Foamed Bitumen and Cement Stabilized Mixes
characteristics and pavement design

DISSERTATION

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To my wife,

Shaghayegh and our lovely daughter Aseman

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Abstract

Considering the rising concerns of the transportation sector in case of negative environmental effects from the construction or rehabilitation activities and more efficient use of the resources, different solutions and approaches have gained attention. Stabilization technology is one of the methods that can be adopted to produce materials with enhanced characteristics, lower production energy, higher rates of recycled products, and therefore to decrease the negative environmental effects. By utilizing foamed bitumen and cement as the binders, a composite product can be produced from different types of granular parent materials. Foamed bitumen and cement stabilized material (FCSM) has a higher bearing capacity than its parent material, lower moisture sensitivity with a balance between flexibility (resulted from bitumen) and rigidity (resulted from cement). It can be integrated into the pavement section with the aim of faster construction, higher share of recycled aggregates, lower production temperature and therefore lower emissions.

Understanding the material's behavior is necessary to be able to get the best out of mentioned advantages and deliver optimum characteristics based on the requirements of each project. In the meantime, the utilization of the material is limited in Germany mainly due to the lack of national behavioral data. Among other parameters, the amount of two binding agents (foamed bitumen and cement) plays a big role in the characteristics of the resulting material. This research tried to take a deeper look into this material and assess the effect of these two binders on its mechanical and performance characteristics and integrate them into the existing national pavement design analytical approach. The research approach was to apply as much as possible the existing methods and available knowledge in Germany.

In the first part, samples were produced with different combinations of bitumen and cement content but the same parent material and mix gradation. Indirect tensile tests in static and cyclic modes, at different temperatures and frequencies, were performed to assess the strength, stiffness and response of the mixes to cyclic load (fatigue). Comparing the stiffness master curves of different mix combinations together showed that the effect of cement on increasing the stiffness is much more than the bitumen. It was shown that the Poisson's ratio amount is an important factor in case of applying indirect tensile mode for the stiffness tests. Poisson's ratio was determined at different temperatures for all mix combinations and its effect on the stiffness master curves was assessed. Comparing the master curves with the reference hot mix asphalt showed a lower temperature dependency of these mixes. Results of the multi-step stiffness tests showed that besides the temperature and loading rate, material's response is also affected by the level of stress (or strain) in the test. Results of multi-round stiffness tests revealed that after the first round, a state of stiffness resiliency forms in which the stiffness is only temperature-dependent and is valid till the maximum experienced strain level. These results showed that the material behavior is a combination of a bituminous bond and granular unbound materials. Fatigue tests results showed a good correlation between the ratio of cement to bitumen and the slope of the fatigue line. It is recommended to keep the cement content low (normally 1% and not more than 1.5% of the dry mass of aggregates). Based on the test's results of this research, bitumen amounts higher than 3% can lead to a resulting material that the fatigue can be taken as the primary failure mode.

In the second part, the gained knowledge was used for developing an analytical structural design for pavements with FSCM layer. A model was developed to consider both behaviors of the material. Its parameters can be determined by using the stiffness master curve and

multi-step stiffness test (one temperature and frequency) results. The model was applied to determine the stiffness at different temperatures and horizontal strain levels as the input parameter into a linear elastic pavement design software. A shift factor was also determined to link the laboratory fatigue results to the pavement life. By considering the results of structural designs with different material models, a three-level structural design method was developed based on the level of the available material data and the required accuracy from the design.

By utilizing the findings and developed methods in this research, it is now possible to produce FCSM mixes, prepare specimens, test them, determine the needed input parameters and perform the structural design of the pavements with this construction type in Germany. These are the foundation for further research and developments on this material and construction type in Germany.

Kurzfassung

In Anbetracht der zunehmenden Bedeutung negativer Umweltauswirkungen von Bau- und Sanierungsmaßnahmen und die effizientere Nutzung der Ressourcen im Verkehrssektor haben verschiedene alternative Lösungen und Ansätze an Aufmerksamkeit gewonnen. Die Stabilisierungstechnologie ist eine der Methoden, die zur Herstellung von Materialien mit verbesserten Eigenschaften, geringerem Energieaufwand bei der Herstellung und höherem Anteil an recycelten Produkten eingesetzt werden kann und somit negative Umweltauswirkungen verringert. Durch die Verwendung von Schaumbitumen und Zement als Bindemittel kann ein Verbundprodukt aus verschiedenen Arten von granularem Ausgangsmaterial hergestellt werden. Schaumbitumen und zementstabilisiertes Material (FCSM) hat eine höhere Tragfähigkeit als das Ausgangsmaterial, ist weniger feuchtigkeitsempfindlich und bietet ein ausgewogenes Verhältnis zwischen Flexibilität (aufgrund von Bitumen) und Steifigkeit (aufgrund von Zement). Es kann Verwendung finden, um eine schnellere Bauzeit, einen höheren Anteil an rezyklierten Zuschlagstoffen, eine niedrigere Produktionstemperatur und damit geringere Emissionen zu erreichen.

Das Verständnis des Materialverhaltens ist notwendig, um das Beste aus den genannten Vorteilen herauszuholen und optimale Eigenschaften auf der Grundlage der Anforderungen des jeweiligen Projekts zu erzielen. Derzeit ist die Nutzung des Materials in Deutschland vor allem aufgrund des Mangels an nationalen Erfahrungswerten begrenzt. Neben anderen Parametern spielt die Menge der beiden Bindemittel (Schaumbitumen und Zement) eine große Rolle für die Eigenschaften des resultierenden Materials. Im Rahmen dieser Forschungsarbeit wurde versucht, dieses Material genauer zu untersuchen und die Auswirkungen der beiden Bindemittel auf die mechanischen und leistungsbezogenen Eigenschaften zu bewerten und sie in den bestehenden nationalen Analyseansatz für die Dimensionierung zu integrieren. Der Ansatz bestand darin, so weit wie möglich die bestehenden Methoden und das in Deutschland vorhandene Wissen anzuwenden.

Im ersten Teil wurden Proben mit unterschiedlichen Kombinationen von Bitumen- und Zementgehalt, aber dem gleichen Ausgangsmaterial und der gleichen Mischgutabstufung hergestellt. Indirekte Zugversuche im statischen und zyklischen Modus, bei verschiedenen Temperaturen und Frequenzen wurden durchgeführt, um die Festigkeit, Steifigkeit und das Verhalten der Mischungen bei zyklischer Belastung (Ermüdung) zu bewerten. Der Vergleich der Steifigkeitsfunktionen verschiedener Mischgutkombinationen zeigte, dass die Wirkung des Zements auf die Erhöhung der Steifigkeit dominanter ist als die des Bitumens. Es wurde gezeigt, dass die Höhe der Poissonzahl ein wichtiger Faktor bei der Anwendung des indirekten Zugmodus für die Steifigkeitstests ist. Die Poissonzahl wurde bei verschiedenen Temperaturen für alle Mischungskombinationen bestimmt und ihr Einfluss auf die Steifigkeitsfunktionen bewertet. Der Vergleich der Hauptkurven mit dem Referenz-Heißmischasphalt zeigte eine geringere Temperaturabhängigkeit dieser Mischungen. Die Ergebnisse der mehrstufigen Steifigkeitsprüfungen zeigten, dass neben der Temperatur und der Belastungsrate auch die Höhe der Spannung (oder Dehnung) im Test die Materialreaktion beeinflusst. Die Ergebnisse der mehrstufigen Steifigkeitsprüfungen zeigten, dass sich nach der ersten Stufe ein Zustand der Steifigkeitsnachgiebigkeit einstellt, bei dem die Steifigkeit nur von der Temperatur abhängt und bis zum maximalen Dehnungsniveau gültig ist. Diese Ergebnisse zeigen, dass das Materialverhalten eine Kombination aus bituminösem Verbund und körnigem, ungebundenem Material ist. Die Ergebnisse der Ermüdungstests zeigten eine

gute Korrelation zwischen dem Verhältnis von Zement zu Bitumen und der Steigung der Ermüdungslinie. Es wird empfohlen, den Zementgehalt niedrig zu halten (normalerweise 1 % und nicht mehr als 1,5 % der Trockenmasse der Zuschlagstoffe). Basierend auf den Versuchsergebnissen dieser Untersuchung kann ein Bitumenanteil von mehr als 3 % zu einem Material führen, bei dem die Ermüdung als primäre Versagensart angesehen werden kann.

Im zweiten Teil wurden die gewonnenen Erkenntnisse für die Entwicklung eines analytischen Dimensionierungsentwurfs für Beläge mit FSCM-Schicht verwendet. Es wurde ein Modell entwickelt, das beide Verhaltensweisen des Materials berücksichtigt. Seine Parameter können anhand der Steifigkeits-Hauptkurve und der Ergebnisse von mehrstufigen Steifigkeitsprüfungen (eine Temperatur und Frequenz) bestimmt werden. Das Modell wurde angewandt, um die Steifigkeit bei verschiedenen Temperaturen und horizontalen Dehnungsstufen als Eingabeparameter in eine linear-elastische Belagsplanungssoftware zu bestimmen. Außerdem wurde ein Verschiebungsfaktor bestimmt, um die Laborergebnisse zur Ermüdung mit der Lebensdauer des Belags zu verknüpfen. Unter Berücksichtigung der Dimensionierungsergebnisse mit verschiedenen Materialmodellen wurde eine dreistufige Dimensionierungsmethode entwickelt, die auf dem Niveau der verfügbaren Materialdaten und der erforderlichen Genauigkeit der Dimensionierung basiert.

Durch die Nutzung der Erkenntnisse und entwickelten Methoden dieses Forschungsprojektes ist es nun möglich, FCSM-Mischungen herzustellen, Probekörper zu präparieren, diese zu prüfen, die erforderlichen Eingangsparameter zu bestimmen und die Dimensionierung von Fahrbahnaufbauten mit dieser Bauart in Deutschland durchzuführen. Dies sind die Grundlagen für weitere Forschungen und Entwicklungen zu diesem Baustoff und dieser Bauweise in Deutschland.

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List of Abbreviations

ASTM: American Society for Testing and Materials

Ave.: Average amount

BASt: Bundesanstalt für Straßenwesen (German Federal Highways Research Institute)

BSM: Bitumen Stabilized Material (Asphalt Academy definition)

CBM: Cold Bituminous Mixes. It refers to all the mixes made with the cold process and the binder is bitumen. Cement can be used as the second binder in these mixes.

CMC: Compaction Moisture Content

CV: Coefficient of Variations

DCP: Dynamic Cone Penetration

ER: Expansion Ratio of the foam bitumen in the foaming test of the bitumen

ER graph: Energy Ratio graph based on the fatigue test results

ESAL: Equivalent Single Axle Load

FCSM: Foamed bitumen and Cement Stabilized Material

FGSV: Forschungsgesellschaft für Straßen- und Verkehrswesen (Road and Transportation Research Association)

FWD: Falling Weight Deflectometer

HL: Half-Life of the foam bitumen in seconds

HMA: Hot Mix Asphalt

HVS: Heavy Vehicle Simulator

IDT Modulus Test: Indirect Tensile Modulus Test. The test is Static.

IDT Strength Test: Indirect Tensile Strength Test. The test is Static.

ITS: Indirect Tensile Strength. This is the result of the IDT Strength Test.

ITFT: Indirect Tensile Fatigue Test.

ITSM: Indirect Tensile Stiffness Modulus. The test is Dynamic. The loading can be different, haversine with rest or sinusoidal.

LCA: Life Cycle Assessment

LCCA: Life Cycle Cost Analysis

LTPP: Long Term Pavement Performance

LVDT: linear variable differential transformer

M.%: Mass Percent

MMC: Mixing Moisture Content

OMC: Optimum Moisture Content

OMMC: Optimum Mixing Moisture Content

PN: Pavement Number in TG2 (Asphalt Academy) pavement design method

RAP: Reclaimed Asphalt Pavement. Some researchers (especially in Europe) use the Reclaimed Asphalt or Recycled Asphalt (RA) aggregate with the same word.

RLPD: Repeated Load Permanent Deformation test. It can be with or without confinement.

SN: Structural Number in AASHTO93 pavement design method

STDEV: Standard deviation

UCDM: Unconfined Dynamic Modulus. The stiffness modulus is determined from the cyclic unconfined compressive test.

UCS: Unconfined Compressive Strength with the static test.

Chapter 1

1 Introduction

This chapter states the existing problem, the need for this research, the aims of that, and the outline of the chapters.

1.1 Problem Statement

Continuously increasing demands and expectations from transportation sector in combination with limitations arising from global issues like environmental and consumed energy concerns, climate change, lack of different types of resources, and increasing demand on availability and reliability of transportation systems have posed a big pressure on different stakeholders in this area. Among different transportation modes, roads as the oldest and the one with bigger share has been always in the center of notice. Considering different components of a road, pavement is the one that has the most effect and interaction with the mentioned issues which has led to continuous seeking of new ideas, methods, and approaches by different pavement experts. Some of them can be listed as:

1- Enhancing special characteristics of existing materials (composite materials)
2- Faster construction/rehabilitation methods (material and pavement type viewpoint)
3- Increasing the share of recycled products in new materials
4- Lowering energy demand and emission level

Among different solutions, stabilization technology is one of the methods with the potential to be utilized and adopted for producing materials that can satisfy the mentioned requirements. Stabilization is generally defined as *a process by which the inherent properties of pavement (and earthwork) materials are altered by addition of a stabilization binder or granular material to meet performance expectations in its operating, geological and climatic environment* [1]. Different types of binders can be applied for stabilization. The most conventional ones are bituminous and hydraulic binders. Bitumen can be applied in two forms: bitumen emulsion and foam bitumen. By using foamed bitumen and cement together as the stabilizing binders, it is possible to produce a composite material from unbound granular mixes. The resulted material has the advantages of cold bituminous mixes (which are mentioned under items 6 and 7) plus some extra potentials:

1- As a composite material, compared to granular material, it has higher bearing capacity and less moisture sensitivity, compared to cement stabilized mixes, it enjoys strength characteristics but also remains flexible; hence it is relatively more fatigue-resistant [2] and also it has no shrinkage cracking issues. Compared to HMA, it is less temperature-dependent therefore lower rutting potential, and has higher air voids which can reduce reflection cracking [3].

2- Compared to bitumen emulsion mixes, this material can be integrated into pavement types with the aim of fast construction. It can be laid and compacted as base layer (in relatively thick layers), overlaid and be opened to traffic shortly after compaction to minimize traffic and construction delays. It also allows for an extended construction season (paving period) as the process is less dependent on weather conditions during construction, compared to hot mix asphalt or other stabilization methods [4, 5, 6, 2].
3- It is possible to apply foam bitumen in cases that the in-situ moisture of the material which is going to be stabilized, is high (near to saturation point). As bitumen emulsion contains water, in such situations it is not applicable.
4- Compared to bitumen emulsion, foam bitumen doesn't contain chemical emulsifying agents and therefore, has negative environmental and hazardous effects.
5- It is easy to produce foam bitumen and doesn't need special knowledge like bitumen emulsion production.
6- The method has the possibility of utilizing recycled materials (different types, especially reclaimed asphalt pavement) with higher contents than the conventional rates in hot recycling methods (nearly up to 100%) [7].
7- Compared to hot production processes, lower energy consumption and also minimum atmospheric pollution [5, 6, 8, 9, 10].

Considering the mentioned possibilities and potentials of the cold mixes with foam bitumen, understanding the material's behavior is the key to achieving the mentioned advantages and delivering optimum characteristics based on the requirements of each project. The utilization rate of the material in Germany is limited and mostly for low-traffic roads. In the meantime, two guidelines are available for cold recycling (known as M KRC and M VBK) [11, 12]. One is for in-place and the other is for in-plant production methods which are not covering all aspects related to mixture production and characterization. The allowable range of binding agents are wide, the assessment tests are limited and there is a catalogue for pavement design for low traffic levels. There are also some documents from different sources (contractors, consultants, and state road offices) [13, 14, 15] which are mainly based on the mentioned two guidelines but tried to address some open parts i.e., methods and instructions in mixture production, tests acceptance limits and the structural design. Considering them plus some research projects performed in this field during the last 20 years in Germany [16, 17, 18, 19, 20, 21, 22] and comparing them to the volume of international research and experience on this material, the amount of national research, especially in case of foamed bitumen mixes is very limited in Germany. Lack of updated standards, guidelines and instructions to manufacture specimens, assess different material's characteristics (physical, mechanical and performance) to prepare inputs for pavement design in combination with inadequate reliable performance data on long-term behavior, resulted in limited and recently no application of this technology and material in Germany. Considering the above-mentioned concerns (which are or will shortly become on the top priority list in Germany too) and the potential advantages of these group of materials (which have been proved internationally), fundamental systematic research is necessary to be able to provide the needed tools for efficient application of them in national level.

1.2 Research goals

Considering the mentioned points, the main question is to see if this group of materials has the value of being applied in the German road network as a construction or rehabilitation material and as a potential solution for some of the mentioned issues? To be able to answer it, the first step is becoming more familiar with the material's behavioral aspects to dimension and assess different pavement scenarios with them.

The increased application of the cold recycled / stabilized layers with foamed bitumen requires research on different characteristics of these materials to propose an acceptable analytical dimensioning method for pavements that incorporate them. Many factors can influence the mechanical and performance characteristics of these mixtures. Some are the origin of the aggregates, type and properties of binding agents, amount of them, moisture content, curing method and compacted density [4]. Because of these variables, it is highly challenging to develop a methodology to analytically predict the performance of such mixtures. This work did not attempt to develop any complicated performance prediction model for the foamed bitumen cold stabilized mixtures, but it was tried to deal with main issues relating to mechanical and performance characterization of these mixtures to be able to define input parameters for the structural design of the pavements with foamed bitumen stabilized layers.

Among the above-mentioned affecting parameters, the amount of two binding agents (foamed bitumen and cement) plays a huge role in the characteristics of the resulting material. The main goal of this research is to look deeper into this issue and assess the effect of these two different binders on mechanical and performance characteristics of foam bitumen-cement stabilized mixtures and then to integrate them into an analytical pavement design procedure. As there is no national guideline for production and testing of these materials in the meantime, therefore besides the main goal, it was also tried to address some of the open questions in this regard by considering their effect. The results from the research will contribute to a better understanding of the material's characteristics and will pave the way for dimensioning of different pavement scenarios with this material as a base layer. Having a design method to calculate the remaining life is the key to a better assessment of different scenarios by applying different LCCA and LCA methods.

To be able to decrease / eliminate the effect of parent material's type on the results, virgin aggregates were selected for this research. The ranges for bitumen and cement were selected to cover the proposed amounts by existing national guidelines and also to be sure that different limits of behavior change are covered too.

1.3 Thesis outline

Chapter one explains the existing problem and the need for this research. The second chapter is a literature review mainly focused on different topics of cold mixes with foam bitumen as the main binding agent. Considering the wide range of topics, the ones which were relevant to the research scope and needed during different parts of that were selected. Material components of the foamed bitumen mixes and their effect on its different characteristics, tests' types and their specifications, important points related to mixture production and specimen's preparation (samples compaction and curing) and topics about structural design of pavements with foam bitumen stabilized or recycled layers are the subjects in this chapter. Chapter 3 describes the research approach and later the materials used for mixture production. It explains the applied procedure during this research for mix production and preparation of the tests' samples then continues with the tests' specifications and plans. Different amounts of bitumen (in foamed type) and cement were selected for mix production (2.5, 3.5 and 4.5 M.-% bitumen and 1, 2 and 3 M.-% cement). Marshall compaction was used as the compaction method and 2 different curing methods were applied (a long standard and a short fast method). Indirect tensile tests were selected as the testing methods. Tests were performed with static and dynamic loading modes at different temperatures. The results are depicted and discussed in chapter 4. Starting with volumetric characteristics and continuing with mechanical and performance tests results. The effects of different material and test-related variables like binding agents' contents, temperature, curing method, load type, its level and history on the stiffness of the mixes and their damage behavior under the cyclic load actions are assessed. This chapter provides an insight into the material's mechanical and performance behavior. Chapter 5 explains an analytical design approach for pavements with foam bitumen stabilized base layers. Based on the findings from the previous chapter, a procedure was developed to combine the results of stiffness-temperature tests and multi-step stiffness tests to prepare the input for the structural model. The model considers both temperature and stress / strain dependency of the stiffness. The method was applied to perform pavement designs for different traffic levels with the mix combination that the fatigue could be accepted as its primary failure mode and then the effect of considering a temperature-dependent stiffness model for the foamed bitumen layer on the results was assessed. Based on the results, a three-level structural design method was developed for the pavements with foamed bitumen layers. Chapter 6 summarizes the important points and the findings of this research and recommends some topics for future research on this material.

Chapter 2

2 Literature Review

Considering that in the meantime no specific standards or guidelines on different aspects of cold mixes with foamed bitumen are nationally available, this literature review should cover different aspects that are important for this research. These are from preliminary issues like the classification of this material, production of the samples in the laboratory to different tests and pavement dimensioning viewpoints. It was tried to first define the important related topics and then classify them in a way that they have the most coherence together.

The chapter will start with a general group of mixes that are produced with the cold process with two different binders (bituminous and hydraulic) to provide a better view of these materials and will continue with different classifications. Then different characteristics and topics will be discussed. Considering the availability of a huge amount of data about these materials with a vast variety of different characterization methods and testing specifications, it was decided to remain focused on foamed bitumen mixes as much as possible. Therefore, the only important topics which were considered most relevant to the research issues are presented in this chapter. The aims of this literature review are:

- To assess different viewpoints among the researchers on this material.
- To perform a survey on different methods and instructions for producing, testing and assessing different characteristics of this material in the laboratory.
- To identify the effects of the material components on its characteristics.
- To survey the pavement dimensioning methods with this material.
- To find the open questions and gaps that need more research.

For this thesis, this review will pave the way to define the research methodology and to try to address some open issues, it also will help in producing more representative test specimens, utilize appropriate test methods and structural design approaches. It can also be used as a base document for the planning of later research programs in this field in Germany. Different sources were used during this review from published papers, university theses, research reports to standards, manuals and guidelines from researchers and road-related offices of different countries to span different viewpoints.

2.1 General

2.1.1 Stabilization and Cold Recycling

The term Stabilization comprises a range of construction processes aiming to overcome the inherent limitations of materials to make the best use of existing locally available resources. These available resources can vary from natural or processed virgin materials up to the ones produced from demolishing processes (recycled aggregates from asphaltic layers, previously hydraulically bonded layers or from unbound layers) and even by-products of other industries (like slags from the steel industry). So, these techniques are not only for virgin materials in new construction but also applicable for materials recovered from existing pavements. Applying this technology in pavement construction and maintenance activities is widely accepted as a cost-effective method of improving long-term performance and reducing whole-of-life costs of modern and heavily trafficked pavements [1, 23].

Pavement recycling comprises a range of techniques by which materials from an existing deteriorated pavement are reprocessed with the aim of producing a new construction material [3]. Based on the production process, recycling methods are classified into two big groups of hot and cold methods. The main difference between these methods is in the temperature level of the aggregate (host material). In cold methods, the aggregates are in ambient temperature and mostly with field moisture state. On the other side during hot methods, the process is done at temperatures above 140 °C. Between these two temperature limits, two other methods have been defined too. Warm recycling technology for temperature ranges above 100 °C and semi-warm recycling technology which applies ranges under 100 °C. lower temperature production has the two big advantages of lower fuel consumption and minor emission rates.

Considering the definitions of stabilization and recycling, During cold recycling, reclaimed aggregates (or materials) are stabilized by applying stabilizing agents. Therefore, stabilization is a general concept that covers cold recycling too. A wide range of stabilizing agents are currently in use worldwide to improve different characteristics of the existing materials during cold recycling and stabilization projects. The most common ones can be categorized broadly into two types of cementitious (hydraulically) and bituminous [1, 23, 7]. Cement, lime, and a combination of these materials with fly ash, ground granulated blast furnace slag are the most cementitious stabilizing materials [23]. For cold processes, bitumen is normally applied in two forms of emulsion or foam [3]. Cementitious stabilizing agents promote rigidity whereas bituminous ones tend to make a relatively flexible material. Cemented material is prone to shrinkage that manifests as block cracks in the layer and when subjected to repeated loads reflecting to the upper layers too; whereas bitumen bound materials are relatively flexible (temperature-dependent characteristics) with tending to deform under repeated load at higher temperatures. However, tensile stresses develop in the lower portion of all layers constructed from bound material under repeated loading, causes them to suffer from fatigue failure. To cover the weaknesses of each group, in most of the cases, a combination of these two binding agents is applied.

Water is another component that is used during the cold recycling and stabilization process. It is added to homogenize the mix, facilitate mixing, hydration of cementitious binder, and aid compaction. After the end of compaction, water should leave the mix to let it gain strength (the process known as curing) [3]. Compared to its parent material before treatment, the produced material will enjoy increased bearing capacity, moisture resistance and better performance characteristics which can be applied as a base layer in a pavement system.

The process of cold recycling/stabilization in the field can be done either “in-plant” by hauling material to a central depot where it is fed through a mixing unit or “in-place” by using different machines designed for mixing binding agents with parent material [7].

2.1.2 Foamed bitumen mixes

To be able to mix the bitumen with cold (and moist) aggregates, it is necessary to reduce its viscosity. Bitumen emulsion and foamed bitumen technologies are two methods to achieve that. Bitumen emulsion is suspension formed from tiny bitumen droplets in water [24]. These droplets are held in suspension with the help of an emulsifier. An emulsifier is a surface-active compound that coats bitumen droplets and gives them an electric charge thus preventing the bitumen droplets to rejoin together. Based on emulsifier type (and the charge of the bitumen droplets resulting from that), there are two main emulsion types: cationic emulsions (positively charged) and anionic emulsions (negatively charged). Bitumen emulsion is produced by blending the water phase and hot bitumen in a colloidal mill. The water phase is prepared as the first step in a special tank by mixing water, emulsifier, acid (or alkali) and other additives. When the emulsion is mixed with aggregate, the bitumen droplets are attracted to the aggregate particles, triggering the setting or breaking process of the emulsion [25]. Slow set cationic (stable grade) emulsions are used for cold recycled / stabilized mixes. Some countries like South Africa, use anionic types too [24]. Base bitumen category, emulsion grade, breaking rate and its compatibility with the aggregate mix are the points that should be noticed during the selection of the appropriate bitumen emulsion [24].

Another way of reducing the viscosity of bitumen is to make foam from that. The first foam bitumen was produced and patented in 1928 by August Jacob from Darmstadt in Germany [20] but the efficient foaming technology was first developed in 1956 by Csanyi who injected steam into hot bitumen which caused the bitumen to foam [26]. The original steam process was impractical for in-place foaming operations, but it was suitable for asphalt plants where steam was readily available. The technology was later modified by Mobil Oil in Australia in the late 1970s [27] by adding a mist of cold water (instead of steam) into a stream of hot bitumen in a low-pressure expansion chamber. The patent expired in the 1990s, since then the technology has been improved with the design of various simplified bitumen foaming systems and efficient mixing processes. In the current technology, a small quantity of water is injected into hot bitumen at temperatures between 160 °C and 180 °C. Water evaporates and the vapor forms numerous tiny bubbles causing instantaneous expansion (foaming) of the bitumen to about 15 to 20 times its original volume. The system developed by Wirtgen [7] in the mid-1990s injects both air and water into hot bitumen in an expansion chamber (Figure 2-1).

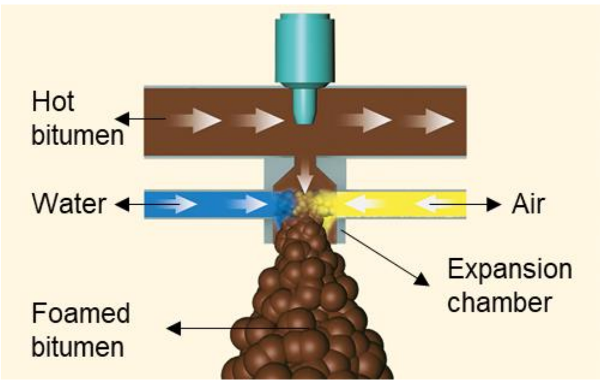


Figure 2-1: Foamed bitumen production [7]

Foaming process increases the surface area of bitumen and significantly reduces its viscosity which improves its mixing and dispersion potential when cold and moist aggregates are used.

During the mixing process, foamed bitumen is selective in its dispersion throughout the mineral aggregate mix, showing preference in adhesion to the finer particles (i.e., fine sand region and smaller). The moisture in the mix prior to injection of the foam assists in the dispersion of the binder during mixing [4]. As the foamed bitumen collapses during mixing, only a limited number of bitumen droplets are attached to the larger aggregate particles resulting in a partial coating of them. The bitumen and fine fraction form a mastic which is squeezed through the coarser fraction skeleton by the compaction and will form the base of bituminous bonds which gains strength by time and exclusion of water (curing). Because of the inclusion of 2 different types of binders plus water and the cold process of production (which affects the dispersion and characteristics of the mastic), resulted material is a composite type with unique characteristics spanning between granular material on one side (especially at its early-life stage) and hot asphaltic or hydraulically bond mixes on the other side. Interaction and integration of many parameters especially the type and amount of binding agents and the characteristics of parent material will determine which side may become dominant. This potential opens the opportunity to be able to design and produce well-balanced mixes which have the positive characteristics of each type based on each project's demands.

Looking to literature, Fu proposed a model for material's microstructure during his Ph.D. work [28]. He identified the formation of 3 solid phases in a foamed bitumen stabilized mix as:

1- *Aggregate skeleton, which is formed by coarse fraction particles of the mix's aggregates;*
2- *Bitumen mastic, which is mainly the finer fraction particles bounded by bitumen droplets;*
3- *Mineral filler phase, which is the fine particles not bonded by bitumen during mixing.*

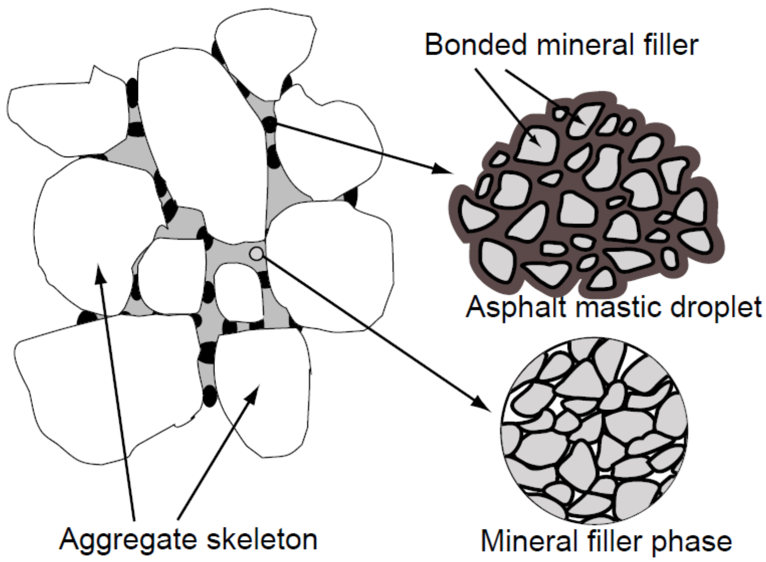


Figure 2-2: Conceptual illustration of foamed bitumen mixes microstructure by Fu [28]

He mentioned that in low contents of cement (around 1%) an independent phase is not formed but the cement will be dispersed into the mineral filler phase and will increase the strength and stiffness of the mix. Looking at this model and considering some extra points:

1- The cement will also affect the bitumen mastic phase properties in different ways,
2- Increasing the amount of cement can form a complete state of bond in the mineral filler phase and also noticeable effects on the bitumen mastic phase,

3- The Mastic phase is also a composite phase itself as foamed bitumen doesn't completely coat the fines in this phase,
4- The size and distribution of mastic and mineral filler phases are not homogenous through the aggregate skeleton phase,
5- The inclusion of water during the curing process and moisture during the service life leads to different responses based on the state of each phase.

These points show how much it is complicated to interpret and predict the response and behavior of the foamed bitumen mixes.

2.1.3 Classification of cold bitumen-cement mixtures

Different countries use different types of cold-produced mixes. They are selected mainly based on the purpose of applying the material (i.e., expected or needed behavior), project's regional climatic conditions, limitations dictated by characteristics of the material that is being treated, binding agents' availability and the production technology. Knowing these different types is necessary for better understanding and synthetizing different methods and reported tests' results. Cold bituminous treated materials (recycled / stabilized) exhibit to a certain degree, characteristics of granular materials, cement-treated materials and hot mix asphalt depending on the mix proportion of binding agents (bituminous and hydraulically) and characteristics of the host material. The effect of variation of bitumen and cement content on the resulted material characteristics can be explained more easily with the help of Figure 2-3 from Asphalt Academy (TG2) [29].

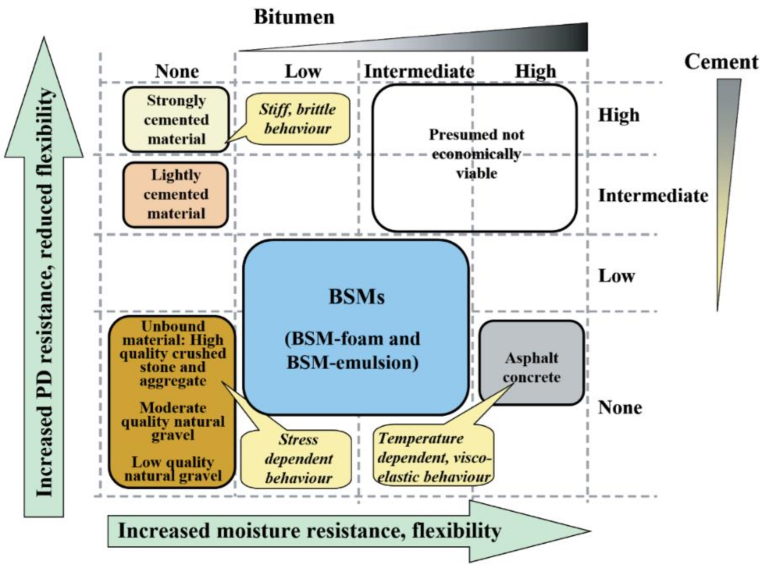


Figure 2-3: Conceptual behavior of pavement materials [29]

As can be seen in Figure 2-3, a variety of different materials can be produced by changing the binder contents. One approach is to increase the material's stiffness and bearing capacity by applying high contents of cementitious binder which results in higher resistance against permanent deformation, moisture and temperature sensitivity but has disadvantages of more rigidity, therefore fatigue and shrinkage crack sensitivity. On the other side is the changes from increasing the bituminous binder where higher binder contents are used to improve flexibility and fatigue performance, the material will have more load rate and temperature dependency (i.e., visco-elastic behavior), like hot mix asphalt [30]. A good balance between these two types of binders can lead to a mixture that can satisfy the needed characteristics in each project. Considering the above-mentioned general concept, different classifications and material names can be seen in the literature. Some are based on rational ranges of each binding agent or the ratio of them together; the others are more precise and have added the results of some tests into the classification process too. In the next paragraphs, some of the important ones are mentioned.

Based on Asphalt Academy guideline (TG2) [29], bitumen stabilized mixes (BSM) are defined with the quantities of residual added bitumen (emulsion or foamed bitumen) typically doesn't exceed 3% (by mass of dry aggregate). In many situations, cement (or hydrated lime) is also added to the mix. The cement content should not exceed 1% (of aggregate mass) and not exceed the percentage of the bitumen, (i.e., bitumen to cement ratio should always be greater than 1). If this ratio (B/C) is less than one, then the material should be considered a cement-treated material. Based on different indicators (page 17 of the guideline), 3 different classes of BSM form 1 (the higher quality which is for higher traffic) to 3 (the lower quality for low traffic classes) have been defined in the guideline. According to Godenzoni et al. [31], B/C around 1 can be the border to have more cement-treated material behavior.

Liebenberg and Visser [32], added some ranges of bitumen and cement contents for low, intermediate and high levels in the above-mentioned conceptual figure (Figure 2-3). In case of cement contents, 0.5-1.5% as low, 1.5-3.0% as intermediate and 3.0-5.0% as high. In case of bitumen contents, 0.5-2.0% means low, 2.0-4.0% intermediate and 4.0-7.0% high. They also proposed a classification based on the results of both UCS (unconfined compression strength) and ITS (indirect tensile strength) tests.

Grilli et al. [33] defined four different kinds of cold-recycled materials as:

1- *Cement-treated materials (CTM)* without any bitumen.
2- *Bitumen stabilized materials (BSM)* like South African specifications.
3- *Cement-bitumen-treated materials (CBTM)* with the residual bitumen contents ≤ 3% and cement contents ≤ 2.5%. These mixtures have considerably higher cohesion and stiffness than the BSM. The basic idea is to start from a cement-treated material and add bitumen emulsion to reduce the cracking susceptibility. They have mechanical behavior close to bituminous mixtures with fatigue as the main failure criteria.
4- *Cold asphalt mixtures (CAM)* with high bitumen contents (more than 3%) and possibly with limited addition of cement. Although they are significantly different from traditional HMA, their mechanical characteristics can be assessed with similar approaches.

This classification is based on bitumen emulsion as the bituminous binder. Foamed bitumen has different coating characteristics and therefore the ranges for the cement and bitumen may be different.

German guideline for in-place recycling (M KRC) [11], proposed the bitumen and cement ranges of 2.5-5.0 M.-% and 1.0-3.0 M.-% for foam bitumen and cement mixes. It also classifies the mixes as bitumen-dominant when their 28-day elastic modulus (determined with the static indirect tensile test at 5 °C based on the explained method in the guideline's annex Nr. 6) is 3,000 to 7,000 N/mm^2 and as hydraulically-dominant when it lays between 7,000 to

12,500 N/mm^2. In the other guideline for in-plant recycling (M VBK) [12], the amount of cement is proposed between 1 to 2 M.-% with a minimum B/C of 2.5:1.

CoRePaSol project report D1.1 (page 15) [34], summarized the binding agents' contents in different European countries. Foamed bitumen ranges from 0.9% to more than 4% and cement from 1 to 5% can be seen in the report. It was later used by Mollenhauer (the report “Potentials for application of cold bituminous bound mixtures in Germany” [16]), rearranged and classified with respect to their binder type and contents as:

1- *Cold Asphalt (CA)*, which is prepared with bitumen emulsion and all the aggregates are coated completely with the bitumen. To reach this state, high residual bitumen contents of more than 6% are needed.
2- *Grave Emulsion* (GE), which is prepared with bitumen emulsion. The difference with cold asphalt is that bitumen builds a mortar that coats the filler and fine fraction of aggregates. The coarser fraction is bounded with this mastic but not continuously. The residual amount of bitumen in these mixes is normally more than 3%.
3- *Bitumen-stabilized material* (BSM), in which both types of bitumen (emulsion and foam) are used, and the main characteristic is that the bitumen only covers the fine fraction, and the resulted material is semi-bound. The range of residual bitumen is 1-3% and the cement is maximum to 1% as defined in the TG2 guideline.
4- *Bitumen-cement-stabilized materials* (BCSM), the main difference of this group with BSM is by increasing the amount of cementitious binder (more than 1% to 3%), beside the non-continuously bituminous bonds, rigid bonds also form in the material too.
5- *Sealing cold recycling materials* (SCRM), increasing the binder contents increases the size of coated aggregates and leads to a bonded state in the material. This potential can be used to seal recycled road materials with hazardous components (i.e., tar contaminated materials).

Summarizing the literature and above-mentioned examples first, it can be seen that different names and abbreviations have been adopted in each country with different ranges of binding agents which makes it impossible to define a classification with a unified naming system. So, it is important when referring to a mix, not to rely just on the naming and abbreviation of the author/s but to consider the definition of that too. In this research, the general name of <u>cold bituminous mixes</u> or <u>bitumen stabilized mixes</u> will refer to the whole group of the material and in the case of foamed bitumen as the bituminous binder, <u>foam bitumen mixes</u> will be used. The general name of <u>bitumen-cement stabilized mixes</u> (also <u>bitumen-cement cold mixes</u>) refers to the group of material with both bitumen types (foamed and emulsion) as the primary and cement as the secondary binder, the host material will be either virgin, recycled or a mixture of them. The general name of <u>foam bitumen-cement stabilized material</u> (FCSM) or <u>foam bitumen-cement mixes</u> will be used for the case that the foamed bitumen is the bituminous binder. All the mixes that will be mentioned later in this research have been produced under the cold method definition except one which is mentioned separately.

Second, it is not possible to define concrete borders for the amounts of each binding agent regarding the changing point of the resulting material’s behavior. Only relaying to the existing experiences in the literature, it seems that 1 to 1.5% cement is the amount which more than that, the cementitious bonds start to become effective in material’s behavior. Lower than this amount the effect of cement can be considered as mostly increasing the moisture resistance and higher than 3% cement seems to shift the material to a more bound cemented state. In the case of bitumen, the issue is more complicated because not only the type of bitumen (foam or emulsion) but also the type of host material (and its different interaction with each bitumen type), becomes more effective too. According to the literature, higher amounts of bitumen in the form of emulsion bitumen may lead to the coating of the coarser aggregate fractions (residual amounts of more than 6%). Considering the non-continuous state of the bonds in intermediate combinations of cement and bitumen (1 to 3% cement and 2.5 to 4.5% bitumen), especially in foamed bitumen mixes, it is not possible to draw a precise definition of the material's behavior. In this range, more research is needed to systematically assess the combined effect of these two different binding agents on mechanical and performance characteristics of the resulted mixtures to be able to define methods to determine them and use them in dimensioning of pavements containing them.

2.2 Material's components and their effect on its characteristics

Foam bitumen and cement mixes are composite materials consisting of different components whose contents, characteristics and interactions together, affect the resulted mixture's different characteristics. Unlike the hot mix asphalt that has only bitumen as the binding agent, these mixes enjoy two different types of binding agents plus the existence of water which changes at different stages during the life of the material; it leads to the formation of different bonds in aggregate and filler skeleton. Considering the interaction of these binders and the wide variations of parent materials makes their characterization very complicated. This section aims to look at these components and their effects on the resulted mix characteristics based on the existing gained experiences mentioned in the literature.

2.2.1 Aggregates and filler

Looking to the literature [30, 8, 35, 36, 37, 38, 39, 20], different types of aggregates spanning from virgin (natural or crushed), to recycled ones (from unbound, asphaltic or hydraulically bounded layers), and by-products (like Slag) and even sometimes locally available material (like Coral in coastal regions) can be stabilized with foamed bitumen. It is clear that the source and type of aggregates affect the resulted characteristics of the mix, but no reference has however specified a type of rock that is not suitable for foamed bitumen treatment. This indicates the low dependency of the foamed bitumen coating to the particle charges of parent material, unlike the bitumen emulsion, where the rock type has a greater influence on the selection and formulation of that as a binder. The granular or mineral mixture before treatment with foamed bitumen is named as *parent material* in the literature.

As foam bitumen in the cold process coats the fine fraction of the aggregates (smaller than 2mm), the amount, gradation and characteristics of this fraction are important factors. The stability of foamed bitumen mixes is more affected by aggregate interlock than the binder's viscosity [40]. This behavior is different from what is seen normally in hot mix asphalt and explains why these mixes are less temperature susceptible than HMA even at the same bitumen contents. The amount of filler (smaller than 0.075 mm or 0.063 mm) is one of the factors which has the most notice in the literature, A minimum need of 5 M.-% of smaller than

0.075 mm particles has been proposed in most of the references [29, 11, 7, 1]. In the case of 100% RAP aggregates, it is always hard to get enough amounts of fines (especially 5 M.-% filler). It is claimed that foamed bitumen droplets have enough energy to heat and adhere to the existing bitumen in RAP, so in case of 100% RAP, the 5% fines requirement can be waived [39, 7]. The positive effect of filler fraction increase is up to a limit because high amounts of filler have the risk of not being coated with foamed bitumen (and also with cement) and to remain as free unbound particles resulting in lower moisture resistance [41]. Maximum particle size and the amount of coarse fraction in parent material plays an important role in achieving acceptable stable skeleton and density in the laboratory and field after compaction [7]. Saleh [42] used two different sizes (40 and 20 mm) of virgin aggregates with the fine amounts around 5% and 7%, stiffness tests results showed the coarser gradation had higher stiffness.

In the “Report on available test and mix design procedures for cold-recycled bitumen stabilized materials” (report D1.1) from the project CoRePaSol [34], there is a table (on page 8) which summarizes the requirements on the material for cold recycling in 8 different countries. The most important fraction in all of them is the filler (smaller than 0.063 or 0.075mm) part which varies from 2% (absolute minimum in case of 100% RAP material) up to 20%.

There are recommended gradations in different guidelines and researches. Asphalt Academy (TG2) [29], suggests a range for foam bitumen mixes and has mentioned that minimization of the VMA (voids in mineral aggregate) is particularly important in the fraction of aggregates smaller than 2.36 mm (sand fraction). Applying the Fuller method with the n equal to 0.45, to determine the passing percentage from each sieve, is proposed. This tight packing will allow for a maximal interlock between the granular material particles and also provides more contact points between them and the dispersed bitumen droplets. It has shown better mechanical characteristics too [4, 43, 44]. According to German guidelines (M KRC and M VBK) [11, 12], fine particles contents are:

- For the in-situ method, with foam and cement, the fraction smaller than 0.063 mm between 3% and 12% and the fraction smaller than 2 mm, ≥ 25%.
- For the in-plant method, smaller than 0.063 mm part, 4 to 9% and the between 0.063 mm and 2 mm, 20 to 30%.

It is important to note that when the gradation of the parent material is not suitable, normally it will be justified by adding appropriate amounts of missing fraction sizes from available other types. Parent material’s gradation can also be used as an indication to determine the range of optimum binder content as a start point in mix design [29, 7].

Besides the mix’s gradation, other physical parameters like the surface characteristics of the aggregates (broken or natural) plasticity, strength indicators are extra parameters that can be checked during the process of assessing the suitability of the parent material [4, 29, 40].

In the case of RAP as the parent material, there are two important points: first, the particles are already coated with an old bitumen and second, a part of them are clusters formed from primary aggregates, still connected together with the old bitumen (A small amount of these clusters will be separated during the production of the new mix with the energy of the mixer). Considering these two mentioned points, the particles have different surface characteristics which lead to different aggregate packing characteristics therefore, different behaviors are expected. Also, based on the state of the old bitumen, the response of these clusters will be different under traffic load and seasonal changes (temperature and moisture) which affects the total response of the material based on their share in the mix. Looking to the literature, incorporation of RAP and increasing its share in the parent material generally leads to higher ITS amount but lower density (and air voids content), fatigue life and resistance to permanent deformation. But in the case of stiffness, different responses are observable in mixes with foamed bitumen [30, 45, 46, 47, 48]. Based on the results of dynamic modulus and dynamic creep tests on cold recycled mixes with foam bitumen and different sources of RAP, Kim, et al. [49] reported that the stiffness is not only dependent on the amount of foamed bitumen but also on the amount and state of the “residual bitumen” in the RAP (residual bitumen here means the extracted bitumen out of the RAP). Coarser RAP materials with low amounts of residual bitumen contents were more resistant to the fatigue cracking but the finer RAP materials with higher amounts of harder residual bitumen were more resistant to the rutting. They also mentioned that the optimum foamed bitumen amount is not affected by the amount of residual bitumen of the RAP but with the stiffness of that. The stiffer residual bitumen, the higher amount of the optimum foamed bitumen [50]. Godenzoni, et al. [51] assessed the effect of RAP by testing mixes with different RAP contents (70, 50 and 0%) and concluded that the complex dynamic modulus is not only dependent on the amount and rheological properties of the foam bitumen but also to the amount of the RAP and its thermo-rheological properties too. Considering the complexity of the issue and the number of different interacting parameters which are influencing that, to compare the effect of parent material type on characteristics of FCSM mixes (physical, mechanical and performance related), a multi-dimensional analysis approach can be adopted [52].

2.2.2 Foam bitumen

The subject of bitumen foaming and its characteristics has been studied by many researchers and practitioners [4, 35, 53]. In the most applied one, foamed bitumen is characterized by two primary properties: expansion ratio (ER) and half-life. The expansion ratio is defined as the ratio of the maximum volume of foam relative to its original volume. It is somehow a measure of the viscosity of the foam and will determine how well it will disperse in the mix. the half-life is a measure of the stability of the foam and provides an indication of the rate of collapse of the foam. It is calculated as the time taken in seconds for the foam to collapse to half of its maximum volume [7]. Jenkins [4] developed the concept of a “Foam Index” to measure the combination of expansion ratio and half-life. Saleh [35] proposed a direct measurement of foam viscosity over the first 60 seconds of foaming with Brookfield rotational viscometer and the mean value as an index to optimize the foam.

Different factors that affect the Foam properties can be summarized as [3, 7]:

- Type and/or chemical composition of bitumen,
- Foaming temperature (base bitumen temperature),
- Foaming water content,
- The pressure of bitumen, air and water,
- Additives added to bitumen,
- The temperature of the mixing chamber

From the literature, a wide range of paving grade bitumen types (50/70, 70/100, 100/150, 160/220) can be used to produce foamed bitumen. Climatic conditions influence the selection of bitumen [34]. Different bitumen types behave differently when subjected to foaming process [54]. Producing a foam with desired characteristics, not only needs a bitumen with appropriate foaming potential but also the foaming conditions should be well optimized. Softer bitumen grades have better foam properties, but harder grades seem to provide thicker aggregate coating [55]. Like in HMA, the type of bitumen can affect the mechanical and performance characteristics of the resulted mix, which is independent of the foaming characteristics of that bitumen [56, 48]. Chomicz-Kowalska & Maciejewski, assessed the effect of three different bitumen penetration grades (35/50, 50/70, 70/100) on mechanical properties of foamed bitumen recycled mixes; based on their results stiffer grade led to higher air voids contents but also higher ITS and stiffness amounts [57]. According to German guideline M KRC [11], both bitumen types of 50/70 and 70/100 can be applied for foamed bitumen production.

Water takes heat energy from bitumen during the foaming and when the bitumen has a lower viscosity, bigger bubbles of foam will be produced. To achieve the desired state of viscosity, bitumen needs to be heated before foaming to temperatures above 160 °C [58]. Increasing the amount of foaming water increases the foam volume and the size of bitumen bubbles which results in the increase of the expansion ratio. On the other side, increasing the size of bubbles reduces the surrounding bitumen’s film thickness, making it less stable and resulting in half-life reduction [4, 59]. Therefore, the choice of the optimum amount of foaming water is a trade-off between expansion ratio and half-life. As the water and bitumen are injected into the expansion chamber, by increasing the pressure in the supply lines, the individual particles become smaller. This will increase their contact area which results in a more uniform foam [58]. Opel [22], tested two different types of bitumen (50/70 and 70/100) from 4 different sources and produced foam with different air and water pressures, the results showed that the 5 bar air pressure and 6 bar water pressure led to higher amounts of expansion ratio and half-life in most of the bitumen sources. Sometimes, depending on the type of bitumen, a foaming agent may be added to get superior foaming parameters [9, 60, 3]. Normally the foaming agents are utilized in warm or semi-warm mixes production when higher amounts of half-life and expansion ratio are required.

It is important to mention that foaming doesn’t change the bitumen chemistry [61] but looking at the results of the dynamic viscosity tests on different bitumen types and sources before foaming and after that at different intervals (2, 24 hours and 7 days), reported by Opel [22], shows a small trend of bitumen’s softening because of foaming process. Twagira assessed the issue of the aging of foamed bitumen mixes in his Ph.D. [62], he recommended limiting the bitumen circulation time at high temperatures in the laboratory plant to three hours and the whole process of heating to 8 hours.

There are no upper limits to foaming characteristics and the aim should always be to produce the best possible quality but there are always minimum limits recommended from different researchers and guidelines as helpful amounts to be taken during the foam mix design. Some countries like New Zealand and Australia require relatively high amounts of ER and Half-life (Min. ERs of 10 to 15 times and half-life of 20 to 30 s) [63, 1]. Others like TG2 and Wirtgen, have limits based on the aggregate mix temperature which has been used more by different researchers. When aggregate temperatures are between 10 to 25 °C, they require a minimum ER of 10 and a half-life of 6s. For temperatures greater than 25 °C, the minimum required expansion ratio is decreased to 8 but the half-life remains the same [29, 7]. In Germany, a minimum ER of 10 times and a half-life of 10 s, are required in M KRC [11].

Due to its high affinity for fine aggregate, foamed bitumen selectively mixes with and coats finer aggregate particles [3]. It interacts with the fines and filler to form a mastic material that binds and stabilizes the mix. So, the amount of binder affects the mastic properties and thus the characteristics of the resulting material. An excess of binder can result in a loss of stability while insufficient binder amounts increase the water susceptibility of the mix [64, 6].

The addition of bitumen fills the voids in the mineral aggregate skeleton and decreases the air voids content which leads to an increase of the density and decrease of the maximum density. This trend continues till the point that the addition of bitumen changes the aggregate skeleton and leads to an increase in void contents and a decrease in achieved density. Compaction method specification (type and energy) and the aggregate gradation (Pack) have an influence on the turning point.

Increasing the bitumen content within a foamed stabilized mixture increases the material's moisture and fatigue resistance [65]. The effect of bitumen content on the fatigue resistance is not as sensitive as in HMA mixtures. Small variations will not have a big influence on the fatigue properties [30]. Increasing the bitumen content of the mixture causes more flexibility [29] but an increase or decrease in the mechanical parameters are depending on the interaction of bitumen content with other mix parameters especially the cement content and the parent material characteristics. Looking to the literature [66, 3, 67, 41], ITS amount usually increases with the addition of foam content, but it continues till a point and then starts to decrease. There are also some tests reports that a clear trend was not seen. In the case of UCS results, there is always a decrease of that with an increase of the foamed bitumen which proves the increase of the mixture's flexibility. In the case of indirect tensile stiffness tests, the results show a pick optimum point by increasing the foam content, but the amount of cement affects the pick point [8, 68, 69, 70]. The results of monotonic triaxial tests on foamed bitumen stabilized samples show an increase of the cohesion parameter (C) compared to unbound granular material but increasing the foam content, has no apparent effect on that. The angle of friction (known as φ) decreases with an increase in foam bitumen. The results of repeated load triaxial tests (RLT) indicate that by increasing the foamed bitumen content, the material's susceptibility to permanent deformation will increase especially when no cement is applied as the secondary binder [8, 71, 72]

Considering the above-mentioned results, it seems that the incorporation of foamed bitumen as a binding agent will improve different characteristics of the resulting mix when bitumen contents are within certain limits. Fu [28] assessed the effect of foam bitumen content on the resulted mix and its properties by observing the changes in the size and dispersion of the bitumen mastic phase in the broken face of the samples from indirect tensile tests. Looking at the pictures and the results of image analysis to calculate the area of this phase, shows that by increasing the amount of bitumen, the mastic phase size increases, and it continues till a point which it affects the uniform dispersion and leads to changes in the trends of some test results.

2.2.3 Cement

After bitumen, hydraulically-based bonding agents are the second binder in cold recycled or stabilized mixes. Among them, cement is the most applied one (different types can be used but the most normal one is the Portland cement type I) and after that, lime (mostly used in Australia [1]) has the second position. The addition of cement (or lime) as the second binder to the mix, has some positive influences on the mechanical and performance characteristics of the mix. This section tries to have a look at these effects.

The addition of cement will increase the rate of mix's gaining strength and stiffness which can be seen by a noticeable increase in ITS and stiffness of the specimens with a decrease in their temperature sensitivity [35, 28, 73, 9, 74]. Results of monotonic triaxial tests on samples with 1% cement showed a noticeable increase in the mix's cohesion parameter (C) and little decrease in friction angle (φ) but leads to an overall enhancement of the shear strength [4, 72]. The addition of cement (even at low rates of 1%) improves the permanent deformation resistance (under repeated loads) which can be seen traced in literature by comparing the behavior of mixes under repeated load triaxial tests [8] and also their performance in the field under accelerated pavement test trials [75, 76].

Cement (and lime) enhances the resistance of the mixture against moisture and its seasonal changes during the service life of the material as a pavement layer which has been proved through different laboratory and field assessments [77, 42, 78, 79, 80, 81, 82]. Generally, the ratio of different characteristics (like ITS, Stiffness or shear parameters) at cured or dried state to a moisture conditioned state, is used to assess moisture sensitivity and is normally limited by a minimum value. The main differences are in procedures to achieve the cured (or dried) state and the moisture conditioning of the samples [62, 83, 11, 7, 29, 28, 49].

Looking at the literature, there are some reports on the positive effect of cement on the fatigue life of bitumen-cement cold mixes. Twagira [30] selected two combinations of RAP and crushed virgin limestone (25:75 and 75:25 ratios) with two different types of bitumen (foamed and emulsion) and 1% cement. Results of flexural bending stiffness and 4-point beam fatigue (controlled displacement) tests showed that without cement and at high strain levels, both types of mixes had no significant difference in fatigue life. The addition of a small amount of cement (1%), led to a notable increase in fatigue life. He ranked emulsion mixes as good-medium and foam mixes as good in fatigue performance. Kavussi & Modaressi [84] reported the same behavior. They applied indirect tensile fatigue test to evaluate the effect of cement content on the fatigue life of the emulsion bitumen mixes with RAP aggregates. Based on the results of the test, they concluded that increasing the cement content from 1% to 3%, leads to better fatigue behavior in low strain rates (up to 0.25‰) but has a negative effect at high strain rates. Leandri et al. [85] performed indirect tensile fatigue (pulse load with rest period) on samples with both foamed bitumen and emulsion and 2% cement. They reported that at high strain levels (0.3‰) both mixes had shorter fatigue life compared to HMA but at lower strain levels (0.15‰) the emulsion mixes showed a better fatigue life similar to HMA mixes.

Yan et al. [86], did a series of tests to evaluate the effect of cement content (0, 1, 1.5 and 2%) on the early-age and long-term performance of the cold recycled mixes. Besides the increase of moisture resistance, high-temperature stability and strength of the mixes, they assessed the effect of cement on the low-temperature behavior of the mixes. They reported a positive effect (increasing trend) till 1.5% but decreased at 2%. It seems that cement has a positive effect on low-temperature crack resistance, but the amount may have an optimum point.

Some researchers count the cement as a part of filler, and they replace it with the equivalent percentage of filler [87] others emphasize that the deficit of the fines shouldn't be compensated with cement [7]. As the cement reacts with water and changes from powder (filler type state) to mortar, this assumption is under question.

It is important to consider that besides the positive effects; cement has also a negative impact which is the increase of brittleness. To decide on the appropriate amount of cement, relying only on the results of tests vs. cement content is not enough and other parameters need to be considered. These parameters are partly related to characteristics of the other components of the mix and their interactions with cement and partly to the pavement structure in which the material is been applied. Better comparisons and conclusions can be made after performing structural design with each mix.

2.2.4 Water

Among the other mentioned components in foam bitumen-cement mixes, water is the one that makes a significant difference between this group of materials and other asphaltic mixes (hot mix asphalt). As the appearance of water in the mix is measured by its moisture content, in the literature, it is addressed as "mixes moisture content" too. Water has a multi-functional role which leads to time-dependent changes in microstructure of FCSM. Jenkins [4] summarized different roles of water as:

- *A separator, suspension and distributer agent of the fine particles* which makes them accessible for foamed bitumen droplets during injection.
- *A carrier to transport and disperse the bitumen droplets* during mixing.
- *A lubricant to provide workability* of the mix at ambient temperatures.
- *A compaction aid* to reduce the angle of internal friction during compaction of the mix.

Considering these roles different moisture contents are considered during the production of samples in the laboratory: Mixing moisture content (MMC) which is the moisture content in the aggregate mix when the foamed bitumen is injected and compaction moisture content (CMC) which is the moisture content of the mixture when the compaction starts. Some researchers and standards take one amount for both of these two parameters claiming that in the real field construction, it is not practical to add water in two different stages, but others claim that as the compaction energy in the field is higher than laboratory, it is important to consider two different stages as in laboratory normally higher moisture content is needed for compaction than in the field. The next paragraphs will have a comprehensive look at the literature about this issue.

Referring to the studies on the optimization of moisture content during mixing, the optimum mixing moisture content (OMMC) can be defined as the amount in which the aggregate mix is in the ''fluff point'' state [59, 88]. It is an amount of moisture in which the aggregate mix has its maximum bulk volume. Aggregate mix gradation especially the range smaller than 0.075 mm sieve (filler part) affects the OMMC amount. It is a function of the optimum moisture content (OMC) of the aggregate mix and ranges from 65 to 90% of OMC determined with standard

AASHTO compaction [89, 40]. Wirtgen [7] and Asphalt Academy (TG2) [29] suggest the same range but based on modified AASHTO / Proctor compactions instead of the standard ones and Austroads suggests ranges for both standard and modified compaction methods [1]. Nyatanyi [20], performed density tests on compacted specimens of five different mixture combinations (of different parent material) with different percentages of moisture (ranging from 50 to 80% of OMC), he reported 70% of OMC as an optimum amount for mixing moisture content. Xu et al. [90] went more in detail to limit the wide range of 65 to 90% based on the filler content of aggregates' mix. They assessed the effect of mixing moisture content on bitumen's dispersion and also on mechanical and performance of the compacted specimens (ITS and monotonic 3 axial tests). They recommended a rational range and an optimal amount for mixing moisture content as:

- In mixes with 5-10% filler (<0.075mm) content, a rational range of 70 to 80% of OMC with optimal amount as 75% of OMC.
- In mixes with 10-15% filler (<0.075mm) content, a rational range of 70 to 80% of OMC, optimal amount as 80% of OMC.
- In mixes with 15-20% filler (<0.075mm) content, a rational range of 75 to 85% of OMC, optimal amount as 80% of OMC.

It is important to mention that the above-mentioned OMC is the aggregate mix's optimum moisture content which is determined with the modified AASHTO compaction method.

Fu [28] addressed this issue by considering the bitumen's dispersion. Mixing moisture content affects the distribution of bitumen by influencing the agglomeration states of the aggregate particles, especially the finer fraction. When the amount is high, granular particles form an agglomeration state which could not yield to optimum bitumen dispersion because of the lower surface area per mass exposed to bitumen. If the construction procedure permits adding water between mixing and compaction, then lower amount of mixing moisture content is desired for bitumen dispersion; if not, then a compromise between mixture's workability (for compaction) and bitumen dispersion should be made to select the amount of mixing moisture content. He suggested 75 to 90% of OMC (based on the modified Proctor) appears to be an appropriate range.

Castedo & Wood [91], defined the mixing moisture content plus foamed bitumen, as "total fluid content" in their study and proposed that the "total fluid content" should be equal to the OMC (based on the standard Proctor method) to achieve maximum compaction of the soil-aggregate mixture. Saleh [35], took the same viewpoint but the results were different. Different combinations of mixing moisture and bitumen content were tested with the aim to maximize the resilient modulus of the compacted mixtures. looking at the reported results for all mixes (with different gradation and hydraulically binding agent types), the optimum mixing moisture content (OMMC) was the same as optimum moisture content (OMC) of the aggregate mix, which was determined by the Gyratory compaction method. The reason is that the foamed bitumen droplets can't be considered as a part of the fluid content of the mix.

Wirtgen [7], recommends 100% of OMC of the aggregate mix as the compaction moisture content (CMC) for laboratory-made specimens however the TG2 [29], recommends that the optimum amount of the compaction moisture should be determined for the mixture itself and with vibratory hammer compaction method. In both guidelines after foam injection, moisture content is increased up to the desired compaction moisture content is achieved. Austroads uses the mixing moisture for the compaction too (water is added in one stage) [1]. According to German guideline M KRC, for foam bitumen mixes, the amount of mixing and compacting moisture should be determined with the Proctor compaction method (DIN EN 13286-2) by including an average amount of the cement (1.5 M.-%) in the aggregate mix.

To sum up this subject, it is important to consider the role of moisture in each life stage of the material (from the mixing stage to after compaction and service). Based on the literature it is better to use two different moisture levels in laboratory as mixing moisture and compaction moisture contents. The mixing moisture content (MMC) can be determined based on the OMC of the aggregate (and cement mix) by considering the amount of filler and the compaction moisture content (CMC) can be taken equal to the OMC.

2.3 Material characteristics and theory of experiments

2.3.1 Introduction

The foundation of each mix and pavement design is based on a good knowledge of different characteristics of the materials used in the layers of the pavement. This can be achieved by testing the material; thus the aim of each experiment is to assess one or more characteristics of the material. Assessment is normally performed by measuring or determining an indicator parameter which is in relation to that characteristic. These characteristics can be divided into three different main categories:

1- physical characteristics with indicator parameters like density, maximum density, the air voids content and permeability,
2- mechanical characteristics with indicator parameters like tensile, compressive or shear strengths, toughness (fracture energy or other related parameters) and stiffness and
3- performance and durability characteristics with indicator parameters like moisture (and freeze-thaw) resistance, fatigue life and permanent deformation behavior.

Mechanical characteristics are the load-deformation or stress-strain behavior of the material which are primarily used for the analysis of the pavement structure responses. On the other side performance characteristics aim to determine the modes of failure which are required to relate the responses to the layer’s service life.

Ideally, the in-situ mechanical properties of the material are required for design purposes. As the in-situ testing of full-scale trial sections is not always practical, engineers must rely on laboratory testing therefore, it is important to be able to reproduce as closely as possible the in-situ conditions (i.e., temperature, loading time, stress conditions, degree of compaction, moisture state, etc.) during the laboratory tests. The in-situ stress state of a pavement material element under a moving load is a 3-dimensional state which changes continually with time (as the wheel reaches that point and passes). On the other hand, as the in-situ conditions are continually changing, selecting the appropriate testing conditions is a difficult and challenging task. It is still not possible to exactly reproduce the complex in-situ stress state in a laboratory test. The repeated load triaxial test is a complex test that reproduces the normal stresses state in an element at the time that the moving load is exactly over that element. Considering the state of the stresses in a pavement element during the load passage, besides the normal stresses, there are also shear stresses whose direction reverses with the passage of the load. This phenomenon changes the amount and plane of principal stresses before, under and after the wheel load and after its passage. Back to the laboratory triaxial test, the cyclic load is being produced with a sinusoidal shape loading, therefore it is not possible to reproduce the normal stress regime in combination with the shear reversal [92, 93]. Considering the above-mentioned points, simplified tests have been introduced to reproduce certain aspects of the in-situ behavior with an acceptable level of accuracy. Looking to the literature, different test methods and concepts have been applied on bitumen-cement cold recycled / stabilized mixtures. Considering and discussing all of them is completely out of the scope and interest of this research. The only important point is that the majority of these tests have the background of hot mix asphalt or granular material and almost no specifically defined test boundary conditions are available considering the inherent characteristics of these groups of materials. This section will try to address the test methods which were utilized through this research to identify the important variables and boundary conditions of each test. It will help to decide on appropriate test specifications and boundary conditions for this research.

2.3.2 Indirect tensile strength test

Binding agents (foamed bitumen and cement) bound the granular particles and thus lead to the initiation of tensile resistance in the resulting material. Most of the material characteristics are directly or indirectly related to the state and properties of bounds in the mixture which can be assessed by tensile strength. In the case of FCSM, this process is time-dependent.

As mentioned before, several test methods are used to determine different characteristics of bitumen-cement cold recycled / stabilized mixes. Among them, indirect diametral (or indirect tensile) and flexural beam are the most normal ones for assessment of material's tensile characteristics, modulus and Poisson's ratio. Between these two methods, diametral testing is more applied because [7, 37, 23, 94, 95, 96]:

- It is simple, easy to set up and provides a rapid rate of testing, therefore, time and cost-effective to perform;
- Sample preparation and handling is simple moreover, the samples can be obtained from field cores too;
- It can be applied to determine different material characteristics (i.e., tensile strength, stiffness, Poisson's ratio, fatigue, moisture susceptibility, creep, fracture toughness and low-temperature behavior);
- It can be used with both static and different dynamic loading types;
- Different temperatures are easy to produce as the test setup is not complicated and doesn't need relatively big chambers;
- Compared to the beam fatigue, the results of the tests are less sensitive to the surface condition and compaction planes of the specimen.

Above advantageous has led to frequent employment by researchers as a standard method.

The test was developed in the late 1960s based on the Brazilian splitting test [97] and later (early 1980s) was accepted as a standard by the ASTM [98]. It was introduced in the UK Through the development of Nottingham Asphalt Tester (NAT) and subsequently gained widespread use in Europe [92]. Typically, the test consists of two vertically positioned compressive forces applying a single load parallel to the plane of the specimen diameter which induces a relatively uniform horizontal tensile stress perpendicular to the direction of the applied load with the maximum in the center of the specimen (Figure 2-4).

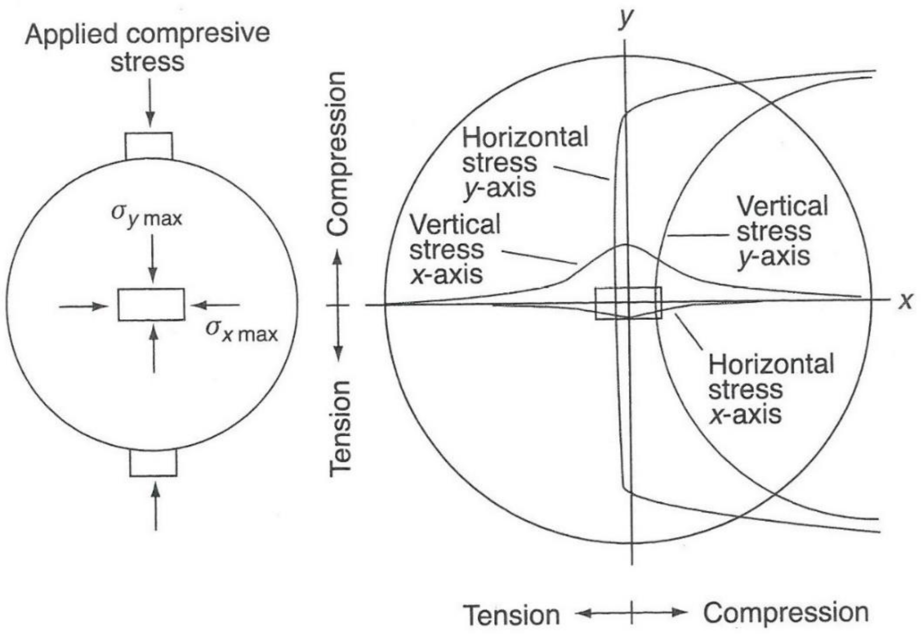


Figure 2-4: Ideal stress state in the specimen under diametrical loading [92]

The force is transferred to the sample through two loading strips whose width and contact curvature is different and selected according to the sample's diameter. With some simplifying assumptions [99], it is possible to determine the stresses at each point in the sample with a closed-form solution [100]. Maximum tensile and compressive stresses will occur in the center:

$$\sigma_{x\,max} = \frac{2P}{\pi Dt}, \quad \sigma_{y\,max} = -\frac{6P}{\pi Dt}$$ **Equation 2-1**

The load is applied through a specified moving rate of the loading cylinder and the peak load at failure is used to determine the indirect tensile strength (ITS) of the specimen [11]:

$$ITS = \frac{2F}{\pi Dt}$$ **Equation 2-2**

ITS, indirect tensile strength (N/mm^2)

F, Peak load (N)

D, diameter of the specimen (mm)

t, height of the specimen (mm)

As the test is simple and fast to perform, the ITS amount is an index to assess the effect of different variables (related to mix components, production or during service parameters) on mixture response and behavior. The index is mostly used for the mix design process as a primary method to estimate the range of optimum binding agents amounts (there are also evidences that sometimes it isn't sensitive to the bitumen content changes [41]). Two main variables of this test are the rate of loading (loading speed) and the test temperature. The specimen can also be tested in dry state or be conditioned already with a moisture conditioning procedure. Deformation rate of 51 mm/Min. is the most used amount but there are tests with stress rate control too [33]. Saleh [42], investigated the viscoelastic behavior of foamed bitumen stabilized mixes by conducting the ITS test at two rates of deformation (51 mm/Min. and 1 mm/Min.). As expected, applying higher deformation (loading) rate results in higher tensile strength and fracture energy. Higher deformations rates (up to 75 mm/Min.) at different temperatures can be used to assess the fatigue cracking behavior of the mixture whereas lower deformation rates (12.5 mm/Min.) are appropriate for assessing the low-temperature crack resistance [28]. The diameter of the specimens is usually 100 or 150 mm and the height varies from 45 to 125 mm (± 5 mm) [11, 3, 28, 101]. Normally the test is done at ambient temperatures (20 or 25 °C) for the tensile strength [7, 29, 35] but different temperatures from -10 to 40 °C were also utilized by different researchers based on their purpose [102, 103, 34]. German guidelines (M KRC & M V-BK) suggest temperature of 5 °C [11, 12].

By measuring the horizontal deformation of the specimen during the test, it is also possible to determine other parameters like strain at break (failure) and elastic modulus too [11]:

$$\varepsilon = 1000 \times \frac{2u \cdot (1 + 3\nu)}{\pi D \cdot (0.274 + \nu)}$$ **Equation 2-3**

$$E = \frac{F \cdot (0.274 + \nu)}{t \cdot u}$$ **Equation 2-4**

ε, strain at failure (‰)

u, horizontal deformation at failure (mm)

D, diameter of the specimen (mm)

v, Poisson's ratio (-)

E, elastic modulus (N/mm^2)

Based on M KRC [11], Poisson's ratio of 0.3 with the load and related horizontal deformation at 45% of failure load are used to determine the elastic modulus.

Looking at the form of cracks that occurs on the sample after its failure, it is possible to get more information on the type of failure modes and the material's behavior. 3 types of failure are reported on ITS tests of specimens in the literature (Figure 2-5).

1- *Clear tensile break (a)*: which is possible to be distinguished by a main clear crack along the diametrical line. The high amount of compressive stresses may lead to small triangular shape failure sections close to the loading strips.
2- *Deformation (b)*: a clear main crack line is not distinguishable, but a combination of cracks can be seen.
3- *Combination (c)*: in this case, a combination of tensile brake lines and larger deformed areas close to the loading strips can be seen.

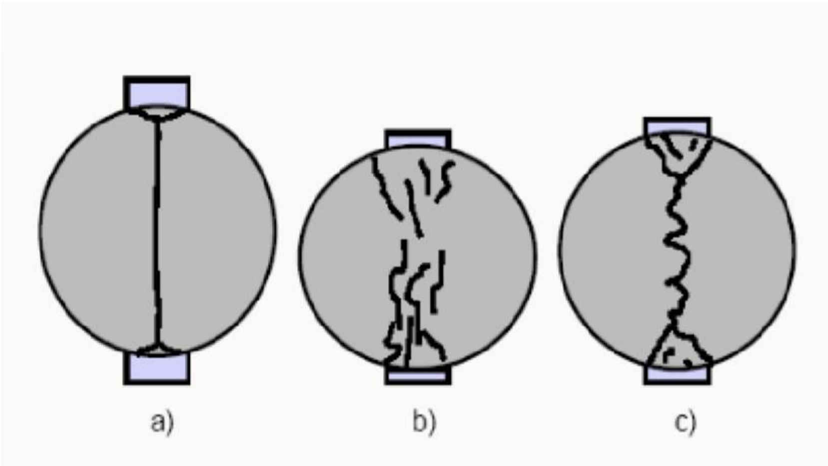


Figure 2-5: Type of failure on ITS specimens [87]

It is important to note that the type and extent of mentioned failures are different in each material regarding the characteristics of its components, their amounts (and ratios) in the mixture and also the production and preparation method. In the case of stiffer mixes with lower temperature dependency, the failure is mostly tensile but in the case of less stiff mixes with higher temperature dependency, it can be a combined type at normal room temperatures. With the inclusion of cement as the second binder, the material's microstructure will change. It affects the type of failure at loading strip areas. At lower amounts of cement, it will be more crushing but by increasing the cement content it will shift to punching. On the other side, the crack propagation model is different in foamed bitumen mixes because of the partial coating of the granular phase. The microstructure model is well defined by Fu in his Ph.D. [28] and later used by Gonzalez et al. [41] to develop a conceptual composite model of the material to interpret indirect tensile strength and fatigue tests' results performed on these mixes (with and without cement).

Besides the advantages of indirect tensile tests (mentioned earlier), compared to the flexural beam or compressive tests, the state of the stress distribution in cylindrical specimens under diametrical loading mode is relatively complicated. Considering the inherent characteristics of foamed bitumen mixes (microstructure components and their homogeneity of distribution through the specimen), other stress distributions may occur in the sample resulting to change of the critical stress points and later the formation of micro-failure zones and their growth model through the specimen. Looking at the pictures of tensile strain (ε_x) distribution in a hot mix asphalt sample under different amounts of vertical load shows that critical tensile strains do not always occur in the central area of the sample [104]. It means that in some rare cases it is possible that the initiation point of the failure isn't in the central area of the sample.

2.3.3 Dynamic (cyclic) indirect tensile stiffness test

Fu et al. [105], investigated different laboratory test methods for resilient modulus of foamed bitumen mixes. They believe that the indirect tensile method overestimates the material's resilient modulus. They suggested triaxial and flexural beam tests instead. It is important to mention that the preparation of specimens for triaxial tests is more complicated as it needs bigger samples and special compaction equipment. The test setup especially for the dynamic tests (to obtain resilient modulus) is also complicated too. In the case of the flexural beam, according to Valentin et al. [106], besides the test setup, the main problem is the preparation of samples. Normally they should be cut from compacted slabs and it is almost impossible to gain beams without broken edges. these defects will affect the test results and may increase the variability of the results.

Dynamic indirect tensile stiffness test has the same test concept as in indirect tensile strength test, but the load type and magnitude are different. The load type is a repeated load well below the failure limit of the specimen. During the test generally, a defined number of load cycles are applied, and the horizontal deformation of the sample will be measured with the external LVDTs, or extensometers glued on the horizontal axis of the sample in the middle area.

Almost all of the road construction materials are not elastic and show plastic deformations under load but if the magnetite of the imposed load is low and repeated, then a state of resiliency can be assumed for them. Therefore, it is important to keep the load level in a range that the measured deformation is acceptably elastic and reversible. The type of load can be cyclic sinusoidal or haversine with resting periods between the pulses. Depending on the type of load and the analysis of measured horizontal deformations, it is possible to determine the instantaneous resilient modulus or total resilient modulus. Stiffness or resilient modulus can be calculated as [92]:

$$S_m = \frac{P \cdot (c_5 - \upsilon c_6)}{\delta_h \cdot t}$$ **Equation 2-5**

S_m, Stiffness modulus (N/mm^2)

P, applied repeated load (N)

t, height of the specimen (mm)

δ_h, resilient horizontal deformation (mm)

v, resilient Poisson's ratio (-)

c_5 and c_6 are constants dependent on the specimen diameter and loading strip width. For specimens with 101.6 mm diameter and 12.7 mm as the loading strip width, $c_5 = 0.2692$ and $c_6 = -0.9974$. As the vertical deformation is not measured in most of the test methods, the value for the Poisson's ratio should be known before.

For hot mix asphalt material, there are standards for the test's specifications and boundary conditions but for bituminous cold recycled / stabilized mixes and especially with foamed bitumen, there are almost no clear specifications. The test's variables are different between researchers. The most important variable is the acceptable range of horizontal strains / stresses for the test.

Horizontal strain level determines the amount of load to perform the test. It should be kept low to be sure that the material remains in elastic zone behavior but also it should be big enough that the horizontal deformation can be measured with an acceptable accuracy during the test. It is also important to consider the range of in-service strains which occur in material as a pavement layer concerning its behavior. For HMA specimens, horizontal strain ranges of

0.05‰ to 0.1‰ (related to horizontal deformations of 5 to 7 µm), are normal [98, 107, 108]. Some researchers used the same range for bituminous cold recycled / stabilized mixes [8, 17, 87, 109, 69, 57, 110] others used lower ranges as they believed that this range may lead to the initiation of micro damages in the specimen as it is less flexible comparing to hot mix asphalt. Horizontal deformation ranges of 2 to 3 µm (for 100 or 150 mm diameter samples) are in the literature [111, 112, 33]. Jitsangiam et al. used 0.03‰ as the target strain [113]. Nataatmadja showed that the results of the indirect stiffness test on foamed bitumen stabilized specimens can vary depending on the test strain level [69] he proposed a horizontal strain level of 0.05‰. It is also possible to use the ITS test results to decide about the stress level for the test instead of strain level. Kavussi & Modaressi [102] took 2 stress levels equal to 15% & 30% of the ITS. Looking to the research of Halles et al. about the effect of stress ratio on the stiffness of foamed bitumen and cement stabilized materials, confirms this range too [114].

Test temperatures of 20 - 25 °C with a loading frequency of 10 Hz or loading time of 125 ms (for pulse loading), are normal for the determination of stiffness modulus. The test temperature must be reflective of the average field temperature. It is possible to perform the test at different temperatures (and loading frequencies) to assess the temperature dependency of the stiffness and also construct the master curve [42, 115, 30, 33, 85, 57]. It is important to note that Foamed bitumen cold mixes are not as temperature susceptible as HMA [116, 117]. As the larger aggregates are not coated with bitumen In these mixes, the friction between the aggregates is maintained at higher temperatures. The temperature dependency decreases by the incorporation of cement. Fu [117] observed that the addition of cement up to 1.5%, doesn't affect the temperature sensitivity of foam bitumen mixes (his specimens had 1.5 to 4.5% foam bitumen with a mix ratio of 70:30 between the RAP and coarse virgin aggregates).

Poisson's ratio

Poisson's ratio is an important parameter that can be determined based on the data of indirect tensile tests and therefore, it is mentioned relatively in detail here. After a short declaration on its importance, the theoretical calculation background is explained; then different approaches and amounts from literature are mentioned. They form the base for the calculation and selection of the Poisson's ratio amounts for the rest of this research. The importance of Poisson's ratio parameter can be seen from different viewpoints. Firstly, it is one of the needed parameters for determination of material's stiffness during the Indirect Tensile Test method. Considering the formula for the stiffness calculation shows that a small change in the amount of Poisson's ratio can lead up to a 25% change in the stiffness modulus [95]. Inappropriate Poisson's ratio amounts can lead to false interpretations of material characteristics. Stiffness (or the stiffness master curve) is also one of the input parameters for pavement modeling in structural design and evaluation purposes and appropriate amounts are desired. Secondly, Poisson's ratio itself is the second parameter in material modeling. It can affect the model calculated responses (like the tensile stresses / strains) at the bottom of bounded layers. Thirdly, it is one of the needed parameters for calculation of the initial strain in indirect tensile fatigue test (ITFT) which is applied for determining the material's fatigue function. Performing a sensitivity analysis shows that the third point is not so critical as the changes in Poisson's ratio have a little effect on the calculated initial strain but the first and second points may be important especially for materials that have some levels of temperature-dependent behavior.

Pavement material's Poisson's ratio is normally determined with different test modes of uniaxial (relaxation or cyclic), triaxial (repeated load [4]) or indirect diametral (static, cyclic, creep or relaxation) [94, 95, 118]. In case of indirect diametral test mode the analytical method is based on solving the stress-strain relationship for plane stress conditions and driving the equation of strain distribution over vertical (along the loading diameter) and horizontal (normal to the loading diameter) axis. By integration of the two strains over the deformation measuring gauge length, it is possible to determine the vertical and horizontal deformations and then by solving these two equations, to have the modulus and Poisson's ratio of the material. In the following text, the mathematical steps and equations of the analytical mentioned method have been described based on the works of Paul et al. in [95]. For plane stress conditions (i.e., σ_z equal to 0), the stress-strain relationship can be written as:

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \end{Bmatrix} = \frac{1}{E}\begin{bmatrix} 1 & -\upsilon \\ -\upsilon & 1 \end{bmatrix}\begin{Bmatrix} \sigma_x \\ \sigma_y \end{Bmatrix}$$ **Equation 2-6**

ε_x and ε_y, horizontal and vertical strains (-)

σ_x and σ_y, horizontal and vertical stresses (N/mm^2)

v, Poisson's ratio (-)

E, Young's (or elastic or stiffness) modulus (N/mm^2)

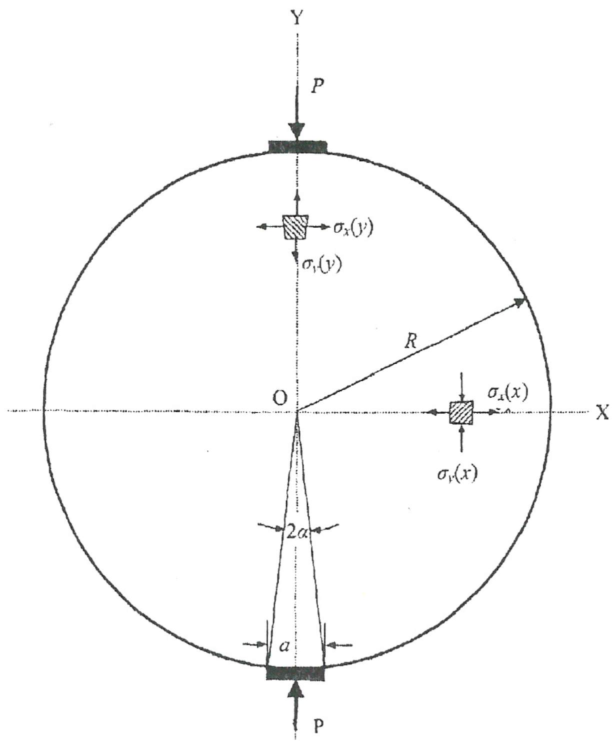


Figure 2-6: Schematic representation of a cylindrical specimen subjected to strip loading [95]

Stresses along the horizontal diameter (OX),

$$\sigma_x\,(x) = \frac{2P}{\pi a t}\left[\frac{(1-\frac{x^2}{R^2})\sin 2\alpha}{1+2\frac{x^2}{R^2}\cos 2\alpha+\frac{x^4}{R^4}} - \tan^{-1}(\frac{1-\frac{x^2}{R^2}}{1+\frac{x^2}{R^2}}\tan\alpha)\right] \quad \textbf{Equation 2-7}$$

$$\sigma_y\,(x) = -\frac{2P}{\pi a t}\left[\frac{(1-\frac{x^2}{R^2})\sin 2\alpha}{1+2\frac{x^2}{R^2}\cos 2\alpha+\frac{x^4}{R^4}} + \tan^{-1}(\frac{1-\frac{x^2}{R^2}}{1+\frac{x^2}{R^2}}\tan\alpha)\right] \quad \textbf{Equation 2-8}$$

and vertical diameter (OY),

$$\sigma_y\,(y) = -\frac{2P}{\pi a t}\left[\frac{(1-\frac{x^2}{R^2})\sin 2\alpha}{1-2\frac{x^2}{R^2}\cos 2\alpha+\frac{x^4}{R^4}} + \tan^{-1}(\frac{1+\frac{x^2}{R^2}}{1-\frac{x^2}{R^2}}\tan\alpha)\right] \quad \textbf{Equation 2-9}$$

$$\sigma_x\,(y) = \frac{2P}{\pi a t}\left[\frac{(1-\frac{x^2}{R^2})\sin 2\alpha}{1-2\frac{x^2}{R^2}\cos 2\alpha+\frac{x^4}{R^4}} - \tan^{-1}(\frac{1+\frac{x^2}{R^2}}{1-\frac{x^2}{R^2}}\tan\alpha)\right] \quad \textbf{Equation 2-10}$$

σ_x (x), horizontal stresses at distance x from the origin (N/mm^2)

σ_y (x), vertical stresses at distance x from the origin (N/mm^2)

σ_y (y), vertical stresses at distance y from the origin (N/mm^2)

σ_x (y), horizontal stresses at distance y from the origin (N/mm^2)

a, width of loading strip (mm)

t, thickness of specimen (mm)

D, diameter (mm)

R, radius = D/2 (mm)

α, angle at the center of specimen subtended by loading strip (°)

P, applied vertical load (N)

It should be noted that the positive sign (+) is used for tension and the negative sign (-) is used for compression. By substituting the stress solutions into equation 2-6, vertical and horizontal strains along the loading diameter (ε_v) and normal to that (ε_h), can be determined.

$$\varepsilon_h = \frac{2P}{\pi a t E}\left[(1+\nu)\frac{(1-\frac{x^2}{R^2})\sin 2\alpha}{1-2\frac{x^2}{R^2}\cos 2\alpha+\frac{x^4}{R^4}}+(\nu-1)\tan^{-1}(\frac{1+\frac{x^2}{R^2}}{1-\frac{x^2}{R^2}}\tan\alpha)\right]$$ **Equation 2-11**

$$\varepsilon_v = -\frac{2P}{\pi a t E}\left[(1+\nu)\frac{(1-\frac{y^2}{R^2})\sin 2\alpha}{1-2\frac{y^2}{R^2}\cos 2\alpha+\frac{y^4}{R^4}}-(\nu-1)\tan^{-1}(\frac{1+\frac{y^2}{R^2}}{1-\frac{y^2}{R^2}}\tan\alpha)\right]$$ **Equation 2-12**

Then, the total horizontal and vertical deformations may be obtained by integrating the above equations over the gauge length (g) which is mounted symmetrically from the center of the specimen.

$$\delta_h = \int_{-g/2}^{+g/2} \varepsilon_h \, dx$$ **Equation 2-13**

$$\delta_v = \int_{-g/2}^{+g/2} \varepsilon_v \, dy$$ **Equation 2-14**

The above equations can be simplified to a general form as:

$$\delta_h = \frac{P}{tE}(a_g + b_g\,\nu)$$ **Equation 2-15**

$$\delta_v = -\frac{P}{tE}(c_g + d_g\,\nu)$$ **Equation 2-16**

Here a_g, b_g, c_g and d_g are the constants that are mainly dependent on the gauge length (g). their values can be determined by integrating the equations. When the magnitudes of these constants for the desired gauge length are obtained, the values of Poisson's ratio and stiffness modulus can be calculated with:

$$\nu = \frac{-c_g - a_g\,\frac{\delta_v}{\delta_h}}{d_g + b_g\,\frac{\delta_v}{\delta_h}}$$ **Equation 2-17**

$$E = \frac{P}{t\delta_h}(a_g + b_g\,\nu)$$ **Equation 2-18**

By measuring deformations along the vertical and horizontal diameters over a specific gauge length, it is then possible to determine the Poisson's ratio of the material with indirect diametral loading method. Paul et al. [95], calculated the constants for different gauge lengths (different amounts of g) for a specimen diameter of 150 mm, height of 85 mm and the loading strip width of 12 mm. Table 2-1 shows the calculated values of the constants for different gauge lengths.

It is possible to apply the constants for any specimen with the same ratio of gauge length to diameter for plane stress conditions.

Table 2-1: Values of constants for determination of stiffness modulus and Poisson's ratio [95]

Gauge Length, g (mm)	ag	bg	cg	dg
37.5 (= D/4)	0.146	0.451	0.490	0.157
75 (= D/2)	0.236	0.780	1.075	0.314
100 (= 2D/3)	0.262	0.911	1.609	0.413
112.5 (= 3D/4)	0.268	0.952	1.970	0.457
150 (= D)	0.272	0.999	4.13	-0.04

Looking at the table, the a_g and b_g constants for a gauge length equal to the sample diameter (the case that the total horizontal deformation is measured using LVDTs at the sides of the sample) are the same as the ones in the stiffness determination equation (Equation 2-5). The important point is when the gauge length is equal to the sample diameter, a big change occurs in the amounts of the two constants c_g and d_g. In this gauge length, the loading strip width has a significant influence on the magnitude of these two constants. It is also important to notice that under the loading strip the amount of compressive stress is relatively high which may cause instability of the elements there and lead to a greater vertical deformation (δ_v). The amounts in the table are calculated for a loading strip width (a) to specimen diameter (D) ratio of 0.08; for the 100 mm diameter specimens and loading strip width of 12.7 mm, they need to be recalculated.

Mirza et al. [94] utilized the same analytical concept (they calculated the constants for 100 mm diameter of the sample and 12.7 mm width of the loading strip) and also studied the effects of specimen misalignment on the amounts of stiffness modulus and Poisson's ratio determined with the analytical method. They indicated that the predicted material properties are relatively insensitive to specimen size and misalignment. The important finding was about the influence of aggregates inclusion on vertical and horizontal deformation measurements. In the vertical plane, the influence of aggregates inclusion may cause errors in deformation measurements outside the central half-radius which significantly affects the value of the predicted Poisson's ratio. In the horizontal plane, the influence doesn't appear to be a significant factor contributing to variations of the horizontal deformation measurements. They concluded that determination of the elastic modulus from horizontal displacements alone has a great potential in providing consistent and reasonable results if an appropriate Poisson's ratio is assumed.

The above-mentioned analytical method plus the results from those two studies, show the complexity of stress state in the indirect tensile test method and the importance of determining correct parameters (especially the related constant parameters and the vertical deformation amount) on the amount of predicted Poisson's ratio. It seems that using smaller gauge lengths by mounting strain extensometers on the sample to measure vertical deformations can lead to the most precise results but in FCSM specimens it is not always possible because of the rough surface of the specimens. Paul et al. [95] had the same problem with lightly cement stabilized granular specimens. They proposed a new on-sample instrumentation set up to measure the vertical deformation. DIN EN 12697-26 annex F [108], which is about the indirect tensile stiffness with cyclic load, suggests installing an LVDT over the loading strip (the upper one) to measure the vertical deformation and thus the Poisson's ratio. Annex C of the same standard is about indirect tensile stiffness determination but with pulse load, there has been mentioned in cases that the Poisson's ratio is not known, amount of 0.35 can be assumed (temperature ranges are from -5 to 25 °C). DIN EN 13286-43 [119], describes a procedure to determine the stiffness of hydraulically bound material with an indirect diametral test method. In this procedure, two deformations are measured. One along the horizontal plane and the other one along a diameter which makes an angle of 60° with that (instead of the normal one which is 90°). Measurements are performed at a force level equal to 30% of the maximum force that the sample tolerates before failure.

Considering the complexity of test setup and calculations to determine the Poisson's ratio for each specimen, guidelines and standards propose ranges or experimental-based equations to determine this parameter. Some of them will be mentioned here as an example.

For bituminous mixes (normally hot mix asphalt), Poisson's ratio varies with temperature and loading time and rate [118]. AASHTO Pavement Design Guide [120] suggests two types of Poisson's ratio values. A constant value of 0.35 or a variable one based on the complex modulus (|E*| in psi) which has the effect of temperature and load frequency reflected in that:

$$\nu = 0.15 + \frac{0.35}{1 + e^{a+b(E^*)}}$$ **Equation 2-19**

Parameters *a* and *b* are -1.63 and $3.84*10^{-6}$ respectively. Islam et al. performed a series of indirect tensile tests in relaxation and cyclic mode at different temperatures and frequencies and proposed -2.041 for *a* and $3.137*10^{-6}$ for *b* [118]. German guideline AL SP-Asphalt 09 [107], proposes the equation from Witczak and Mirza. which relates the Poisson's ratio only to the test temperature T (in °C):

$$\nu = 0.15 + \frac{0.35}{1 + e^{[3.149 - 0.04233*(\frac{9}{5}*T+32)]}}$$ **Equation 2-20**

For cement-treated aggregates, AASHTO [120], recommends values between 0.15 and 0.35 almost the same range is proposed by Austroads (0.1 to 0.3) with a typical value of 0.2 [121].

In FCSM mixes, depending on the amount (ratio) of binding agents (especially, cement) and the type of parent material, their Poisson's ratio can vary from an asphaltic material (variable model especially with temperature) to a cemented bound material (constant model). therefore, these two models can be taken as two limits to assess the Poisson's ratio of these mixes.

M KRC [11], suggests 0.3 for the Poisson's ratio at 5 °C. Mixes are ranging from 2.5 to 5.0% foam bitumen content and 1 to 3% cement content. Austroads [122], suggests the amount of 0.4 to determine the resilient modulus of foamed bitumen stabilized materials at 25 °C with the indirect tensile method. Johannsen and Willmeroth [13] proposed the same amount in the technical specification for dimensioning and production of foundation layers with foam bitumen (EUROVIA). Sunarjuno [87], assumed an amount of 0.35 at 20 °C, for mixes with 2% foamed bitumen, no cement and RAP as the parent material. The same amount was taken by Iwanski and Kowalska [123] at 25 °C and later by Kowalska and Maciejewski [57] at other temperatures (-10 to 25 °C), for mixes containing 2 to 3.5% foam bitumen and 2% cement with 50% of RAP and recycled granular base aggregates (50:50) as parent materials. In another research, they proposed temperature-dependent Poisson's ratio amounts of 0.25 at 0 and 10 °C and 0.35 at 20 °C for mixes with bitumen emulsion (3% residual bitumen) and 2% cement. Kavussi and Modaressi [102], assumed a constant amount of 0.35 for temperatures ranging from -10 to 25 °C. Mixes had 4% emulsion bitumen and different cement contents of 1 to 3% with RAP as the parent material. CoRePaSol project team [70], used following values of Poisson's ratio during their research tests: 0.27 for 5 °C, 0.31 for 15 °C and 0.38 for 27 °C. They also used the mentioned DIN EN 13286-43 method to determine the Poisson's ratio of specimens at different temperatures (-10, 10 and 30 °C) and with different cement and bitumen contents. The results were highly scattered especially at -10 °C (table 18 of their report). It can be because of the difficulty of fixing the LVDS at 60° axis. Valentine [38], proposed amounts from hot mix asphalt for mixes with high contents of RAP and low amounts of cement. Radenberg et al. [17] used the Witczak (AL SP-Asphalt 09) equation (which is for HMA) to determine the amount of Poisson's ratio. Their specimens had relatively high bitumen emulsion and low cement contents. The parent material was RAP with different ages and gradations.

Jenkins [4], performed triaxial cyclic tests on foamed bitumen stabilized samples and showed that Poisson's ratio can be also stress state dependent. This phenomenon occurs when the amounts of foam and cement are relatively low (in materials under BSM classification).

2.3.4 Indirect Tensile Fatigue Test (ITFT)

Pavement materials that are bounded by cement and / or bitumen show fatigue resistance properties when subjected to specific levels of repeated tensile stresses / strains. According to ASTM [124], fatigue is defined as "the process of progressive localized permanent structural change occurring in a material subjected to conditions that produce fluctuating stresses and strains at some point or points which may culminate in cracks or complete fracture after a sufficient number of fluctuations". Research results on foam bitumen-cement mixtures show that mixes with relatively high contents of foam bitumen (> 3.5%) behave in a manner with some similarities like HMA [4, 68].

Over the past 60 years, different test methods have been developed to simulate the fatigue behavior of bituminous road construction materials. According to the mode of loading the most commonly used tests are classified into three groups, simple flexure, direct uniaxial loading and diametral loading tests. According to the stress-strain distribution in the specimen, the fatigue tests are divided into two types, the homogenous type and the inhomogeneous type. If the stress-strain distribution is (nearly) uniform throughout the specimen, the test is called a homogeneous test (like uniaxial loading test). For non-homogeneous tests, such as bending tests and the indirect tensile test, the stress-strain field is not uniform along the specimen and cross section [125].

There are different test methods to assess the fatigue behavior of the bituminous mixes. EN 12697-24, specifies several tests to characterize the fatigue behavior in bituminous mixtures, as follows [126]:

1- Two-point bending test on trapezoidal shaped specimens.
2- Two-point bending test on prismatic shaped specimens.
3- Three-point bending test on prismatic shaped specimens.

4- Four-point bending test on prismatic shaped specimens.
5- Indirect tensile test on cylindrical shaped specimens.

The concept of the indirect tensile test was described before; the difference with indirect tensile stiffness test is the continuation of loading till a defined failure point (specimen fracture or a specified deformation level). Since the time of its development (early 1970's) with the aim to characterize the strength and stiffness of bituminous materials, indirect tensile test was used as a method to assess the fatigue properties of these materials too [127, 128, 129, 130].

During the fatigue tests on HMA samples usually, three failure patterns can be observed [125]: 1) crack initiation at / or near the center of the specimen, resulting in complete splitting of the specimen; 2) crack initiation at the top of the specimen, progressively spreading downward in a V-shape, the arms of which originate from the outside edges of the loading platen; and 3) no real cracking occurs, with the specimen being plastically deformed beyond the limiting vertical deformation.

Like the indirect tensile stiffness test, two types of loading can be used; a sinusoidal cyclic load or a haversine pulse type (with rest periods). The test temperature is normally 20 °C for HMA (in special cases, other temperatures may be applied too).

Twagira [30] has made a literature survey on the fatigue performance of cold bituminous mixes with foamed bitumen and bitumen emulsion, during his Master thesis. He summarized some important points that can affect the fatigue response as:

1- Mixture variables (i.e., binder type and content, aggregate type and gradation, the inclusion of cementitious binders) and compaction method during the specimen production.
2- Load related parameters, such as type and frequency. Cyclic sinusoidal load is the most practical type of loading as it reduces the needed test time. It should be kept in notice later an appropriate and larger shift factor will be required as this loading type underestimates the in-situ fatigue performance. A loading frequency of 10 Hz is a good representative of mixed traffic conditions and also doesn't generate excessive heat in the specimen and therefore is suggested for the fatigue test.
3- Environmental variables, especially the test temperature affects the fatigue response. He found that 5 °C is the critical temperature for beam specimens and applied that.
4- The addition of small quantities of cement led to an increase in the fatigue life of the foamed bitumen mixes compared to the ones with bitumen emulsion.
5- When emulsion or foamed bitumen are applied with higher percentages of RAP (75% was used) a decrease in fatigue life occurred.
6- Based on the results of fatigue tests, he compared the performance of the selected cold mixes (emulsion or foam), with the guidelines developed for HMA. By considering the point that in practice they are thicker than HMA, he categorized the emulsion mixes as good to medium and foamed bitumen mixes as good, in fatigue performance.

CoRePaSol project team stated in their final report (proposed harmonized test procedures for durability characterization, deliverable D2.2) that as it is almost impossible to prepare proper and precious beam test specimens of cold bituminous recycled materials, indirect tensile fatigue test on cylindrical samples is the most suitable method to assess the laboratory fatigue performance. They proposed the EN 12697-24 as the relevant standard with test temperatures of either 10 or 20 °C [106]. More details can be found in their report on fatigue characterization of cold recycled mixes [131].

One important subject is the definition of the sample's fatigue point during the indirect tensile fatigue test and then the method to determine its fatigue life. Looking into the literature, for this type of material three different methods have been used:

1- Decrease of stiffness to 50% (or 30%) of its initial amount [30, 32, 23];
2- By plotting the cumulative transverse permanent deformation vs. loading cycles graph and then the interception method [84, 126]; and
3- By applying dissipated energy methods [17, 131].

Zack & Valentin [131], determined the fatigue life of foamed bitumen mixtures by applying different analyzing methods on the ITFT results. Looking at the fatigue lines shows a high level of variability in the results (independent of the analyzing method) with low values of the regression parameter R^2. Inhomogeneity of aggregate coating when foamed bitumen is used as the binding agent can be the main reason for that. They proposed that the Energy Ratio method is a suitable approach for determining the fatigue life.

It is believed that the slope of the fatigue line is affected by binder type and the mix stiffness [132] so, it can be a good parameter to assess the effect of different binding agents and their content on the fatigue behavior of the foam bitumen and cement stabilized mixtures. Gonzalez et al. [41], utilized the ITFT on mixes with a combination of RAP and virgin aggregates (the ratio around 0.33) with different amounts of foam bitumen (1.25, 2.5 and 3.75%) and 1% cement. They used standard Marshall compacted samples with a nominal aggregate size of 25.4 mm and proposed that performing the ITFT at a constant tensile horizontal stress (250 KPa) can be used as a complementary test to identify the optimum bitumen content when the ITS test's results don't show a clear optimum point.

Another important point in reviewing the literature is the high level of variation in fatigue tests' results (either indirect tensile or flexural mode) of foam bitumen-cement mixes [131, 133, 134, 30, 135]. It is mainly the result of the inherent state of the bonds in the mixture which are not continuous. How to deal with this issue and up to which level the variation is acceptable, are complicated questions which are not been answered till now.

2.4 Sample production

The first step in each research or construction project is the production of the mixture and then the test's samples. It is important that the process be performed right and constantly in the same way to be able to have comparable and reliable test results. Important points related to the mixture components and their effects on the characteristics of the resulted mix were mentioned in sections before, the aim of this section is to introduce key and critical parameters during the production of the mixture and test specimens in the laboratory.

Sample production steps can be summarized as:

1- Dry mixing of the aggregate with cement and then mixing with water (the aim here is to bring the moisture content to the fluff-point);

2- Injection of foamed bitumen in the mix as the mixer is working and continue mixing;
3- Adding the rest of the water and mixing again;
4- Compaction of the samples;
5- Curing the compacted samples based on a suitable method.

The most important parameters are aggregates temperature, mixing process, compaction and the curing method, which will be discussed in the next paragraphs.

After the injection of the foam bitumen into the mixer, the bitumen baubles will exchange heat with the aggregate mix particles. The quality of coating and the size of coated aggregates are in direct relation with their temperature at the time of injection and mixing [4] which later has a direct impact on the amount of determined optimum bitumen and the characteristics of the material [136]. To have comparable results, it is important to keep the temperature of the aggregate mix on a constant and appropriate level for all the mixes during a test program. To get realistic results, it is recommended to select the aggregate mix temperature based on their real field temperature during the construction time. Bowering and Martine [137], recommended a critical temperature range of between 13 °C and 23 °C for the aggregate mix before injection of the foam bitumen.

Foamed bitumen is injected directly from the laboratory foaming plant into the working mixer. Both field production methods (including twin-shaft pugmills and milling-drum mixers) provide enough volume in the mixing chamber plus agitation energy to ensure that the aggregate mix is airborne when it contacts with the foam bubbles. Therefore, it is important that the laboratory mixer be able to represent the same state too. Cazacliu et al. [27], assessed the effect of different process parameters during the production of the foamed bitumen mixtures. They concluded that the twin-shaft laboratory mixer produces smaller mastic particle sizes (between 0.5 to 1.5 mm) compared to the planetary mixer which was twice bigger. It was shown in the study that the decrease in mastic particle size has a positive impact on material cohesion after compaction. They also observed that higher numbers of mixing rotations decreased the size of the mastic particles but after 80 rotations, its effect became negligible. Mixing durations of 20 seconds up to 2 minutes for laboratory made samples can be seen in the literature [7, 41].

2.4.1 Compaction

It is well known that the applied method and degree of compaction affect the aggregate pack interlock in the specimen (can be seen in the broken face pictures [138]) which can change its volumetric and later mechanical and performance characteristics. Abiodun [3] reported that a 4 percent decrease in compaction degree, resulted in about 65 percent decrease in ITS value. As mentioned before, water is a compaction aid by providing the possibility of lower contact resistance in the mix letting the aggregates to move and reorient. Because of the presence of water, the compaction mechanism of foamed bitumen mixes is different from hot mix asphalt. The compaction procedure should rearrange the coarser aggregates and simultaneously force the mortar (conglomerates of mastic and partially coated sands) to be squeezed in between the voids of this rearranged coarse skeleton; forming the different bonds (bituminous and hydraulically) and leading to an appropriate amount of air voids (after evaporation of the water). Normally during the compaction procedure, a certain amount of energy is applied over the fresh mix to reach the mentioned goal. Reviewing the literature shows a variety of different compaction procedures among researchers or in guidelines / standards for foamed bitumen cold mixes. The most common ones can be listed as:

- Marshall compaction [138, 38, 139, 140, 6, 141, 115, 29, 142, 41],
- Modified Proctor (or AASHTO) [7, 13],
- Static pressure compaction [11, 38, 143, 144, 17],
- Gyratory compaction [35, 144, 69, 145, 87, 146, 50],
- Vibratory hammer compaction [29, 147, 148, 149].

There are some uncommon methods like roller compactor (which is used to compact slabs) [17, 112] and vibratory table compaction [149]. The most difference between these procedures is in the type of applying the compaction energy on the fresh mix.

Marshall method (DIN EN 12697-30) imposes compaction energy through impact loads by applying a hammer that falls on the sample from a constant predefined height. The number of the impacts (blows) is the same for each side of the sample. Different number of blows can be used to achieve different amounts of energy (35 to 75 blows per side). For foamed bitumen mixes, 100 mm diameter and 50 to 75 blows per side are common. The advantages of this method are its availability in almost any laboratory, simplicity of the procedure and high rate of sample production.

Proctor and AASHTO methods (standard or modified version) are conceptually the same as the Marshall method as they also apply an impact load via a hammer. They are common for soil and granular mixes (DIN 18127:2012-09) but also for stabilized materials with hydraulic binders too (DIN EN 13286-2). Based on the aggregate size, different mold and hammer sizes have been standardized for them. These methods are also utilized for foamed bitumen mixes but mostly as a reference density.

During the static pressure compaction method simply a vertical load is applied over the sample via a compaction plate but with different procedures. The load can be kept constant for a certain amount of minutes that the water drains out of the mix or can be applied in several cycles till a steady state response is reached. Most of European countries have specifications for this method with the main difference in the amount of applied pressure. CoRePaSol project team (in the report D1.1) [34], compared the mechanical characteristics of samples compacted with different methods (gyratory, Marshall and static pressure) and proposed the compaction pressure of 5 MPa as a harmonization for EU countries; but they didn't make any comparison with field densities. German cold recycling guidelines [11, 12], propose this method with 2.8 MPa pressure which is the lowest among all other countries.

The principle of gyratory compaction (DIN EN 12697-31) is based on the combination of static pressure from the top and rotation of the mold simultaneously. The mold moves in a way that its axis creates a conical rotating surface. With the change of the angle, constant pressure amount and the number of rotations, different compaction results can be achieved. Jenkins [4], Kuna et al. [145] and Valentin et al. [144] (CoRePaSol project team), made good literature reviews on different gyratory compaction procedures.

Kim et al. [50] discovered from their studies that additional foamed bitumen leads to better compaction of the mixtures with the gyratory compaction, but not under the Marshall hammer. Kim and Lee [150] observed that although gyratory compaction can produce higher densities than 75 blows of Marshall, the competed samples exhibited lower resilient modulus values than the Marshall compacted ones. The same results were reported by Nataatmadja [69].

Valentin et al. [144] (CoRePaSol project team) compared the effect of compaction method on bulk density, indirect tensile strength and stiffness modulus of the 150 mm diameter specimens with 4.5% foamed bitumen and 3% cement. They utilized 3 different compaction methods: 75 blows of Marshall, static load press with 5 MPa pressure and gyratory compaction with 600 to 900 kPa vertical pressure and different rotations of 40, 60 and 80. They observed that the Marshall compacted bulk densities were lower than the ones compacted with static pressure. In the case of indirect tensile strength, the same trend with the static pressure compacted samples having the highest amount. In the case of the stiffness modulus, Marshall and static pressure compacted samples were the same and higher than the gyratory compacted samples except for the one which was compacted with the 900 kPa vertical pressure and 80 rotations. Khweir [140] compared the stiffness and density of the cores taken from the site with the samples compacted with different Marshall blows (30 to 75). He concluded that 60 to 75 blows of Marshall lead to samples that are representative of the field compaction.

Jenkins et al. [148], believe that the impact and static compaction methods don't lead to an aggregate orientation similar to what happens in the field where vibratory rollers with variable amplitudes and frequencies are applied. They proposed the vibratory hammer as the best simulator of the field compaction with a high rate of reproducibility. It is possible to apply the compaction with two methods: by fixing the time of compaction or by reaching a predetermined thickness. The second one is more reasonable as the amount of the needed energy is not fixed and depends on the mix properties and the desired compaction level. On the other hand, a reference density is needed already to determine the amount of fresh mix for each layer. Considering the changing behavior of density with time because of the presence of water makes this more complicated to determine the reference density.

It can be seen from the literature that there are a variety of compaction methods proposed and assessed by researchers. When comparing different compaction methods together, the first point is to consider that the amount of binding agents (foamed bitumen and the cement) and the characteristics of the aggregate (especially their type and gradation) can affect the results from each compaction method in a different way. Therefore, a comparison result between two compaction methods may not be valid for other cases. Higher amounts of bitumen can change the viscosity characteristics of the mastic which makes the static pressure based compaction methods to show better results, but lower amounts may shift the mix to the other side which vibratory and impact-based methods may lead to better results (can be seen in the works of Nataatmadja [69]). The other point is that the comparison should be made by considering the amount of energy which is imposed on the mix in each method and always the link between the laboratory and the field (in the case of roller types which are common for compaction of the foamed bitumen mixes), should be considered too.

2.4.2 Curing

Generally, the term curing refers to the process in which a material gains strength and stiffness over time. Mechanical characteristics (strength and stiffness) of foamed bitumen mixes are relatively low after the end of compaction (both in the lab. and the field) but increase over time with curing. As mentioned before one of the main reasons for adding cement is to increase the early life strength of the mix before the full completion of the bitumen bonds. As the laboratory curing method can affect the results of the mechanical and performance tests of the specimens, it needs to be considered during the production of the test specimens.

Considering that the foamed bitumen mixes contain a noticeable amount of moisture before and during compaction, through the curing process this moisture should go out of the mixture to let the bitumen bonds be formed. This happens mainly due to evaporation and in the case of mixes containing cement, partly through its hydration process too. The aim of laboratory curing is to let the moisture of the samples reach a stable content.

Fu [28] looked to the curing mechanism of foamed bitumen mixes by considering different phases of the mix in his Ph.D. As mentioned before, he presented a microstructure model for foamed bitumen mixes consisting of 3 different parts: the aggregate skeleton, bitumen mastic phase and mineral filler phase. The aggregate skeleton is formed of the coarse aggregate fraction which can withstand applied loads mainly due to its inter-particle contacts. The strength of this phase is available after the end of the compaction and is not affected by the curing. Bitumen mastic is a mix of bitumen and fine fractions which is formed during the mixing. It bonds the aggregate skeleton together but the bonding forms through curing as the moisture leaves the interface between the aggregates and the mastic. The mineral filler phase consists of the filler particles which are not coated or bonded during mixing. This phase has a low strength amount after compaction (as it is still wet) but relatively high strength after drying. The inclusion of cement in the mix as the second binder, makes the micro-structure more complicated, as its hydration can affect both the mastic and mineral filler phases' curing and thus strength and stiffness development. Fu believes that in low cement contents (ranges of 1 to 2%), an independent phase is not formed, and cement particles are mostly dispersed in the mineral filler phase. By comparing the fractured face of the samples after its tests, Fu et al. [151] found that in sealed specimens (no water evaporation), the weakest regions were in the aggregate-mastic interface but in unsealed specimens which had the possibility of water evaporation, the weaker region was the bitumen mastic. The same observations were reported by Li et al. [152]. They assessed the fractured face asphalt coverage (FFAC) other specimens at different ages of curing. They reported the increase of FFAC with curing age. This indicates by the curing progress and water evaporation, the adhesion (bonds between aggregates and bitumen mastic) becomes greater than the cohesion (from bitumen mastic) and three therefore the failure path moves more through mastic which results in to increase in FFAC. It is important to notice that the mixes in these researches were without cement. The incorporation of cement into the mix can change the results by affecting not only the adhesion and mastic cohesion but also by creating a new cohesive phase from the boding of the free filler phase. The suction of the residual water is a phenomenon that results in the formation of weak bonds in the mineral filler phase and leads to an increase of stiffness over time [105, 151, 153]. Kuchiishi et al. [154] performed a quantitative analysis and evaluated the role of matric suction on stiffness gaining in cold recycled foamed mixes. They indicated that matric suction increases by water evaporation from the specimen. They validated that by showing the reduction of air voids measured with X-ray tomography. By analyzing the resilient modulus from triaxial tests, they mentioned that besides the other dominating mechanisms, the matric suction can influence the stiffness of foamed bitumen mixes too.

The most important objective of a laboratory procedure for curing is to produce specimens that are representative of the field conditions. Field curing is a complicated process that depends on several factors including day and night-time temperatures (absolute amounts and the gradients), humidity and rainfall levels, wind, layer thickness, applied amounts of bitumen and cement, moisture content of the mixture (before mixing, at injection time and after that), the achieved level of compaction, in-place air voids content, and the drainage characteristics of the beneath layers and the road shoulders [155]. It is possible to classify them as: regionally related, material related, and construction-related factors. Different field monitoring projects show that foamed bitumen layers continue to gain stiffness up to one year after construction even under traffic loads and seasonal changes [65, 156, 157]. These observations of changes in material's characteristics over time, raise the question: "at which state of the time the needed parameters should be determined?" the first and simple approach is to take the material's ultimate cured state. The second approach is to define different curing procedures to reflect different material's life stages (i.e.: just after compaction, short-term or during the curing and long-term or after curing) [20, 122]. The third approach is based on a claim that during the field curing process, the amount of moisture never reaches zero; it decreases to a point till a state of equilibrium is reached in the whole pavement system [158].

Temperature and relative humidity are the two most important parameters which their level and duration, can noticeably affect the loss of moisture and the hydration level of the cement in laboratory specimens. Increased levels of temperature, lead to faster moisture evaporation (fast curing methods) but in mixes with cement contents higher than 1%, longer durations of increased temperature can affect the hydration process negatively as the remained amount of moisture in the sample may not be enough for the cement hydration. If the temperature level is around the softening point of the bitumen, it can also affect the bitumen flow in the sample. Normally higher temperature levels are imposed with lower duration and lower temperatures levels with higher durations. Fu [28] and Twagira [30], performed a good literature survey over different curing temperatures and durations. They range from ambient temperatures of around 20 °C up to higher levels of 60 °C with durations of 24 to 72 hours for higher temperature levels to longer durations of 7 to 28 days for ambient temperature levels. The important points are:

- 60 °C is not appropriate as curing of the samples at temperatures above 50 °C, showing the flow of bitumen.
- Curing temperatures close to the ambient temperature may impair the repeatability and reproducibility of the procedure as controlling the amount of relative humidity can be difficult.

Fu concluded that 40 °C appears to be an appropriate compromise. Asphalt Academy (TG2-2009) [29], Austroads (AGPT-T305) [122] and Wirtgen [7] propose 72 hours as the duration for this temperature level.

In the case of relative humidity, higher levels simulate the early life of the mix in the field so, in most of the methods, samples are kept in the mold or sealed at the first 24 hours. Considering the explained concept of equilibrium moisture content in the field, Jenkins [4], proposed to seal specimens in plastic bags during the curing. The procedure is adopted in the Asphalt Academy guideline (TG2-2009), too [29].

During the CoRePaSol project, the research team surveyed different curing procedures and assessed the effect of curing parameters (temperature, moisture, duration) on strength and stiffness. In their report on *harmonized mix design procedures* (D1.2) [159], they proposed 24 ± 1 hour remaining in mold and then based on the content of cement, for mixes with ≤ 1.5%

cement, curing of unsealed specimen at 50 °C (or 40 °C) for 72 hours. For mixes with > 1.5% cement, curing of unsealed specimens at room conditions (around 20 °C with relative humidity between 40 to 70%) for 14 days, but depending on each country, it can be up to 28 days.

German guidelines M KRC and M VBK have two procedures which are because of the different allowed ranges of cement. Based on M KRC which allows higher cement contents up to 3% for foamed bitumen mixes, after remaining in the mold over the night, the samples are cured for 2 days at a temperature of 20 ± 2 °C with a relative humidity of ≥ 95%, and then the curing continues in room conditions (temperature 20 ± 2 °C and relative humidity of 40 to 70%) for several days (7, 14 or 28 days) according to the test specifications [11]. On the other hand, M VBK which allows the maximum amount of 2% cement for foamed bitumen mixes, recommends only the room curing procedure [12].

This compact literature review on the subject of curing in foamed bitumen cold mixes shows the variety of procedures and the complexity of this issue which is still a research topic in this field. The aim of this section was to mention the important parameters that affect the curing process of these mixtures. Before selection of the appropriate curing procedure, these points should be considered: the relevancy of the method to the field conditions (regional climate and construction factors), the interested life stage of the material (short or long-term), material's special properties (especially the amount of cement), simplicity of the method, its repeatability and reproducibility.

2.5 Structural design of cold bituminous materials

Over the past 70 years, pavement design has gradually evolved from an art to science. Before early 1920, the thickness of the pavements was determined purely based on experience [160]. As experience was gained throughout the years, various methods were developed by different researchers and were utilized in different countries. The introduction of computer-based calculations opened the possibility of more complex models. This section won't introduce all of them, but some relevant ones will be cited to show the trend. Various design procedures can be summarized under two primary categories of Empirical and Analytical methods.

2.5.1 Empirical methods

These methods are based on the analysis of the collected data and performance observations from different test sections or pavements that are under the service. Normally the procedure is improved over time by involving more gathered data. Among these methods, the Structural Number (SN) and the Pavement Number (PN) methods are the most relevant ones for cold recycled / stabilized mixes.

Structural number (SN) design method

The method was first established based on the AASHTO road test results (performed between 1958 to 1960) in the USA and developed for years till the last version known as AASHTO 1993 pavement design method [161]. Thanks to its simplicity, it is used worldwide for the design of all types of pavements. The design equation relates the pavement structure to its service life and is defined as the number of equivalent single axles loads (ESALs) of 80 kN. Other parameters like serviceability and reliability factors have also been considered in this equation too. SN is an abstract that reflects the structural strength of a pavement. It can be converted to actual layer thicknesses by using layer coefficients (a_i) which represent the relative strength of the construction material in each of the pavement's layers. If no drainage effect is assumed, the structural number of pavement is defined as:

$$SN = \sum a_i \,.\, D_i$$ **Equation 2-21**

a_i, layer coefficient of the material in layer i (1/inch)

D_i, thickness of the layer i (inch)

Structural layer coefficients are normally quoted in terms of "per inch'' values which are well known worldwide. It is preferable for countries with the metric system to retain the coefficient with this unit and convert the layer thickness from centimeter to inch in calculating the SN. Required SN for a given combination of reliability level, soil support (M_R), total traffic (expressed in ESALs), terminal serviceability and project's climate data is determined from the equation and compared with the pavement's existing SN. If it is lower or equal to that, then the design is considered to be adequate.

To dimension pavements with cold recycled / stabilized layers by this method, only the layer coefficient of the material is needed to be determined. Layer coefficient value depends on several factors (resilient modulus, underlying support, stress state, etc.). It can be determined by having material characteristics like ITS, E modulus or cohesion and angle of friction [7, 37, 153, 110] or from field experiments data [162, 163, 164, 165, 166, 167]. For foamed bitumen mixes depending on material characteristics, different amounts from 0.18 to 0.39 can be found in the literature [168]. The reliability of the results obtained using this method is dependent on the accuracy of the input parameters especially the selection of an appropriate structural layer coefficient. Besides all the limitations, in the case of recycled materials, it is still improving by the results of different research projects. Some references like the Wirtgen manual [7], recommend the method for structural capacity requirement up to the 10 million ESALs (of 80 kN) but the author believes that it can also be used for higher traffic levels as a quick method to determine the thickness of the layers at the starting phase of design.

Pavement number (PN) design method

The PN method has been developed through an extensive research program in South Africa. Observing different results from two Heavy Vehicle Simulator (HVS) test projects on a trial section [169] was the main reason for reconsideration of the design method mentioned in the first edition of the South African TG2 manual. The research program took almost 5 years to be completed and involved the combination of laboratory work (Stellenbosch University), HVS and LTPP (Long Term Pavement Performance) analysis of 23 pavements [170]. Chapter 5 of the TG2 (2009 edition) [29], explains this method. It is a knowledge-based approach with a reliability of 90 to 95%. The method is recommended for pavements carrying up to 30 million ESALs (of 80 kN). It is similar to the SN method but instead of layer coefficients, it uses the "Effective Long-Term Stiffness" (ELTS) for each pavement material. This procedure includes a comprehensive classification system for the materials in various layers and also considers climate, layer position in the pavement structure and the amount of cover over the subgrade. It applies the modular ratio rule to control the selected stiffness of the material in each layer to ensure that a balanced pavement system is achieved. PN of an assumed pavement structure is determined by summing the product of ELTS value and the respective layer thickness. The calculated number is then used in a curve to determine the structural capacity of the assumed pavement.

2.5.2 Analytical Methods

The common part of the analytical methods is that they all include a calculative approach to determine the critical responses (stress, strain and deformations) in a pavement structure due to applied load (and environmental actions). The relevant values are then linked to the structural capacity (life of the layer or the pavement) by means of transfer functions. The most common procedure in this group is the Mechanistic-empirical approach which is well explained in different literatures [160]. Two important parts of this method are the approach for material model (later pavement model) and then selecting / determining of the transfer function. FCSM is mostly applied as the base layer in a pavement system; based on the design requirements, an asphalt packet with different thicknesses covers that.

Multi-layer linear elastic theory is generally used for analyzing the pavement models, mainly because it is relatively simple and easy to set up. To consider the temperature dependency of the stiffness, it is possible to divide the layer into sub-layers and assign an appropriate stiffness amount to each of them and run the model for different time periods that the temperature gradient can be assumed constant. The same concept can be used to consider the non-linear behavior by subdividing the layer and performing the analysis till convergence of the resilient modulus. Other analytical methods like finite element analysis techniques (FEM) it is possible to consider more complicated material behaviors (i.e., non-linearity, viscoelastic, elastoplastic or anisotropic) or boundary conditions (i.e., three-dimensional models or the bonding state between the layers) [8, 171]. On the other hand, more complicated models require additional material parameters and if they are not determined correctly, the risk of having wrong results will be higher.

Transfer functions are equations that relate the critical relevant responses to the service life of the layers. They can be an empirical-based function, driven from research and / or pavement performance data or can be determined from the results of laboratory performance tests. The first group of transfer functions are ranging from simple equations with easy determinable input parameters to more complicated ones with different calibrating constants which are needed to be determined for each material type and pavement conditions. Before utilizing these transfer functions, their background history must be known to find out their validity range of application. Laboratory transfer functions can be more specific and precise as they can be driven from the same material which is used in the pavement section but linking them to field results, needs to determine appropriate “shift factors”.

To determine which responses are critical and what type of transfer functions (or performance tests) should be utilized, it is crucial to find out how the material behaves and what is / are the failure modes of that. FCSM mixes are very complicated as they are elasto-plastic (because of the non-continuous state of bonds distribution in the mix) with granular type and stress history dependent behavior and also visco-elastic (because of the existence of bitumen) with loading time and temperature dependency [169, 172]. How these behaviors share into the resulted material's dominant behavior is dependent on different factors mainly: the amount of binding agents, the type and characteristics of parent material, its gradation and the resulted pack after compaction and curing, the type of pavement in which the material is utilized and the state of the imposed actions (from loads and weather) on the material during its service.

Early analytical methods for cold mixed materials were developed based on mixes with higher cement contents so, the material behavior models were influenced from cement stabilized materials models [169]. Two different phases were assumed for the mix's life. the first phase is the fatigue phase which is with a considerable reduction in the stiffness (known as Stiffness Reduction Phase). After that, the stiffness is almost constant and the behavior is like granular material which permanent deformation is the main failure mode. This phase is known as Steady State Granular Phase. There are transfer functions for each phase in the literature [32, 58, 173, 174]. The method was suggested in the first edition of TG2 (2002) and the Wirtgen recycling manual (2004). Later researches (mainly made by South African researchers) [169, 175, 176] showed the two-phase performance behavior is under question for mixes with low amounts of bitumen and cement (known as BSM, see section 2.1.3 for the definition).

In the meantime, there are two approaches for the performance behavior of foam bitumen and cement stabilized / recycled mixes. Both approaches have good support from laboratory test results and field observations on trial or real sections (the results of two different field tests in references [168] and [177] as the latest examples). The first approach is more valid for low rates of bitumen and cement. It claims that with these low amounts of binding agents, the material is not fully bonded and acts more like granular material but with enhanced sheer properties (semi-bond material). In this case, the stiffness is stress-dependent and non-linear models are suggested to precisely determine the critical responses. The primary failure mode is permanent deformation. Deviator stress ratio (which is the ratio of the applied deviator stress relative to the maximum deviator stress of the material at failure) method is the general design procedure justified for this approach [7, 58]. It has been proved that the rate of permanent deformation in a bitumen stabilized layer is a function of the deviator stress ratio (DSR) [4, 178]. After determining the DSR from the pavement response analysis and material's shear parameters (c & φ), graphs or transfer functions are applied to determine the allowed number of load repetitions to reach a defined amount of permanent strain in the layer (different transfer functions can be seen in Bierman works [178]). This approach is more supported and applied by South African and New Zealand researchers. Wirtgen proposed the method for traffic levels higher than 30 million ESALs (of 80 kN) [7]. Zhang et al. did a case study on constructed trial sections supported with laboratory tests. Different recycling methods (central-plant and in-situ) with foam bitumen and bitumen emulsion were compared with a standard HMA section. In one of the sections, RAP was treated with 2.2% foam bitumen and 1.5% cement. They reported that the material has viscoelastic temperature dependency, but the main failure mode was permanent deformation [10].

The second approach which seems to be more valid for higher rates of bitumen, claims that the material can be assumed as a bounded material. The stiffness is temperature-dependent but not as hot mix asphalt, and the primary mode of failure is fatigue. Strain at the bottom of the layer is the main critical response used in conjunction with transfer functions to predict the life of the layer and the pavement. As an example, Khweir et al. [179] applied a linear elastic software package (BISAR® 3) to determine the critical strains under the standard axle load of 80 kN and performed indirect tensile fatigue tests in the lab. (Nottingham testing machine with rest periods) to determine the fatigue equation of the foamed bitumen material. Then, he increased the results by a shift factor of 3 to determine the in-situ fatigue life of the foamed bitumen layer from the laboratory results. The same approach was used by Nyatanyi [20] for determining the critical responses under a 130 kN axle load. Then he used the graphs from the Shell pavement design method (SPDM) to determine the allowable horizontal tensile strain for his desired pavement life. Leek [180], performed fatigue test on beams sawn from at least 3 months old pavements at 10 Hz frequency and temperature of 20 °C. He proposed a transfer function based on the results which was justified later based on more test results with recommending a shift factor of 10 to 20 to relate laboratory fatigue results to the field [134]. Twagira [30] used the same loading frequency but 5 °C and proposed a shift factor of 20. A shift factor of 17 to 20 can be found in the van Wijk report [181]. Iwanski & Chomiz-Kowalska [138] utilized the Asphalt Institute fatigue criterion from the Mechanistic-Empirical Pavement Design Guide (MEPDG) to determine the fatigue life of a foamed bitumen stabilized layer. It is important to note that the equation is very sensitive to the air void content of the mix. Pitawala et al. [182] used the Austroads asphalt fatigue equation for the mechanistic design of foamed bitumen stabilized pavement bases. They mentioned that the equation may need some justifications in the case of using foamed bitumen mixes.

A good reference in literature is the Austroads report on “Design and Performance of Foamed Bitumen Stabilized Pavements” [183]. Based on the findings and intensive monitoring of six trial sections with bitumen contents of 2.5 to 4% and with hydrated and quicklime (0.8 to 1%), fatigue cracking is the primary distress mode for foamed bitumen stabilized pavement layers and this needs to be considered in structural thickness design. The report mentions that the fatigue is not considered in South Africa and New Zealand mainly because the South African mixes contain lower amounts of bitumen (less than 3%) and cement (maximum 1%) and also lower amount of fines compared to Australian mixes (based on [1], In Australia, it is common to use foam bitumen contents of 2.5 to 3.5% and hydrated lime contents of 1 to 2% for host materials with up to 20% RAP and the amounts of filler is normally higher than mixes in New Zealand). This difference leads to having higher amounts of mastic in the resulted mixes. The report suggests the utilization of the cured soaked indirect tensile modulus (measured at 25 °C), then adjusted to in-service temperature (weighted mean annual pavement temperature) and loading rate to determine the design modulus. It is recommended to limit the amount of modulus to 2500 MPa to be sure that the mix becomes not too rigid. The thickness design procedure assumes that the material has a sufficient quantity of residual bitumen to produce a bound state (usually a minimum of 3%) and also doesn’t include too much cement or lime (maximum 1% cement or 1.5% lime) to control the mixture’s susceptibility to fatigue. Two distress types should be controlled for the pavement thickness requirements: first, fatigue cracking of the foamed stabilized layer using the horizontal tensile strain at the bottom of the layer and second, rutting and shape loss determined using vertical compressive strain on the top of the subgrade. The report proposes an asphalt fatigue equation (the base of that is from Shell asphalt fatigue equation) to predict the fatigue life of the foamed stabilized layer (eq. 4 on page 51 of the report). The equation is associated with design reliability of 95% and with a shift factor of 6 can be applied for design reliability of 50% (mean predicted fatigue life).

Another good reference is the CoRePaSol report “incorporation of cold-recycled pavement layers in empirical and mechanistic pavement design procedures” (deliverable D3.1) [184]. It starts with presenting some results of different accelerated pavement tests and trial sections and continuous with a review on various empirical and analytical design methods in different countries (USA, South Africa, New Zealand, Australia, Ireland, Czech Republic, Portugal, Spain, France and Germany). Second part focuses on parameters that are normally required during a pavement structural design procedure. Besides the material's stiffness, it addresses fatigue and permanent deformation as the key design parameters. For fatigue failure criteria, two points were mentioned: firstly, the fatigue failure might not be suitable for mixes with low contents of bituminous binder (< 2.5% residual bitumen) and / or hydraulic binder (< 3.0%). Secondly, it was proven that the only viable testing method to assess the fatigue performance in the laboratory, is the indirect tensile method. For permanent deformation characteristics, the wheel tracking test might not be applicable instead, triaxial testing was suggested.

Some researchers claim that as FCSM mixes (with RAP as the dominant fraction of the parent material) showing a relatively high level of variation in bearing capacity characteristics, it is better to consider and apply them as a foundation layer in a pavement section. They suggest the permanent deformation as the primary failure mode which can be controlled with the amount of compressive stress on top of the layer [13]. This idea may ignore the potential use of higher quality mixes in base layers.

2.5.3 Design catalogues

Some countries have developed design catalogues for traditional pavement materials, giving suitable pavement structures and layer thicknesses based on empirical analyses for specific regional parameters. Some examples can be found in M KRC [11] and Wirtgen's handbook [7]. CoRePaSol team (in the above-mentioned report) [184] made a comparison between the German (M KRC) and Wirtgen catalogues for different combinations of traffic (3 classes), sub-base bearing capacity and cold recycled mixture type. By comparing the results for low volume and main roads, the overall pavement thickness is similar for both approaches. In M KRC, thicker hot mix asphalt surfacing layers are applied but in the Wirtgen approach, granular sub-base material is used. They claimed it can be because of tighter requirements for evenness and rutting depth in Germany.

As can be seen from the literature review in this section, there are different design approaches between countries and researchers. It is important to consider each approach and method with its material characteristics as a package and don't mix parts of different methods. For empirical-based methods, more references are available, the level of accuracy is acceptable and they are in the steady progress of improvement. In the case of analytical methods, it is necessary to have a correct understanding of the material's behavior and failure mechanism which is mainly affected by the amount of binding agents. There are still open questions like what are the appropriate laboratory test methods and then justification approaches to get representative input parameters for pavement modeling, determining critical responses and performance evaluation? how to relate laboratory performance results to the field performance and life of the layer / the pavement? Which approach should be applied to deal with tests' results variabilities? Considering the mentioned questions, it seems that analytical methods still need more time to be assessed and validated based on field performance observations.

2.6 Summary

The literature review tried to provide a general view of cold bituminous mixes and focused on different topics related to cold mixes with foam bitumen as the main binding agent. It will be considered as a reference base during the next steps of this research to identify the best practice methods for specimen production, definition of the tests' specifications and to interpret the observations and the results of the tests when needed.

Foam bitumen and cement stabilized mixes are composite multi-phase materials with different components which change over time (curing). By varying the components' ratios, the resulted material may behave in a wide range spanning from enhanced granular (as semi-bound) up to bonded materials (with asphaltic dominant or cemented dominant). A good understanding of material's components and their interactions together is essential for producing mixes with enhanced special characteristics based on the requirements of each project.

The material has a combination of elasto-plastic and visco-elastic behaviors and the resulting dominant behavior will be variable and dependent on not only the components' characteristics and their interaction, but also on the service-state related parameters (temperature, moisture, stress / strain level and history). Sometimes the interaction of components together and with the service environment may activate some hidden unseen characteristics and show different behaviors. Therefore, for studying the effect of a parameter, it is important to fix other variables as much as possible and also to decrease and eliminate the risk of activation and interaction of unknown characteristics.

In the case of parent material, it is possible to stabilize a wide range of host materials with foamed bitumen but there are some parameters as the gradation and the amount of fine fraction which should be noticed. Parent material's type, gradation and morphology (round or crushed) are factors that affect the behavior of the resulting mix. In case of recycled cemented granular, the portion of aggregates with not fully hydrated cement; and in RAP, the state of the bitumen are also important.

Mixture production and samples preparation process affects the material's characteristics and therefore the tests' results. Aggregate temperature, the mixing process, mixer type and also mixing energy are the important parameters that should be considered. Understanding the role of moisture in different life stages of the mix is important. The compaction method should be representative of the field and also satisfy the timing limits and production requirements in the laboratory. The curing procedure should consider the amount of binding agents and the strength gaining process of them and should also ensure that a representative level of long-term moisture is achieved in the sample.

Generally, the test methods applied on FCSM are borrowed from other material types and there are still open questions in case of tests' boundary conditions, specifications, assessment of the results and ranges of acceptance. These open questions have led to some adjustments by different researchers for each test. Therefore, it is important to acknowledge them during the interpretation of the results. The indirect diametrical test method to characterize tensile strength, stiffness and fatigue behavior has been utilized and is the most recommended method by researchers. It has the disadvantage of a complicated stress state which requires some simplifying assumptions for analysis and arises the question that up to which level these assumptions of homogeneity and elastic behavior are valid?

Structural design of the pavements with foamed bitumen stabilized layers is mostly developed based on experimental approaches. Experimental methods have the most references and are in a progressive improvement. They can be used as a tool for control and preliminary design purposes. As the material spans a wide range of characteristics, therefore relying only on the catalogues may not always be economical. Analytical methods have the advantage of using as-produced material's mechanical and performance characteristics in the design process, but they are very limited and still under research and development. One of the main issues is how to extract the required design parameters from laboratory tests' results.

Chapter 3

3 Research Approach, Materials and Laboratory Tests

This chapter describes the research scope and the approach of that, then laboratory tests and their specifications for this research and at the end explains the steps for mixture production and specimen preparation for the tests.

3.1 Research approach

The main goal of this research is to deliver a better understanding of the behavior of foamed bitumen and cement stabilized mixes (FCSM) to be able to integrate them into pavement types in Germany. As the binding agents are one of the key components affecting the behavior of these materials, understanding how they affect the pavement design input parameters is the first step. This research tried to address that by specifying the research tasks. Considering the research goals, findings from the literature review part and the current state of knowledge and experience on FCSM in Germany, the below questions were raised:

1- How to produce and prepare FCSM specimens in the laboratory? Considering the literature review, compaction and curing are the main issues in the production and preparation of the specimens.
2- How to determine the physical parameters of FCSM specimens?
3- How will FCSM respond to the load? how to determine mechanical parameters of FCSM for analysis of the material's response in a pavement model?
4- How does it perform under cyclic loading and what is the effect of binding agents' properties on that?
5- How to perform the structural design of pavements with FCSM as the base layer?

To address each of the above-mentioned questions, the approach was to apply as much as possible the existing available and common knowledge in Germany (i.e., standards, guidelines and instructions) aiming to figure out up to which level they are applicable for this material. In cases that justifications were needed, findings from the literature review were the base to select the best practice method based on available equipment and testing possibilities in the laboratory.

The first two questions were the basis for some preliminary tests and research activities which were planned and accomplished through a Bachelor thesis (see [185]) as a part of this research. The findings and learned lessons from these preliminary activities were the base to produce specimens and determine their volumetric indexes. Questions 3 and 4 made the foundation of the main tests of this research which will be explained later in this chapter. The aim of question 5 is to try to integrate the results into pavement dimensioning instruction.

3.2 Type and range of binding agents

As discussed already (see section 2.1.3) different amounts of bitumen and cement (hydraulic binder) can be applied as binding agents during the stabilization or cold recycling processes. This may result in different material behaviors ranging from enhanced granular material types to bonded mixtures with bituminous or hydraulically dominant behaviors. Characteristics of the parent material, special expectations from the resulted mix and the economical considerations are the primary parameters in selecting the range of binding agents which is normally finalized by a mix design process. The process may differ between countries and that leads to different dominant behaviors in the produced mix. As the research aims to assess the effect of binding agents on material characteristics, a range should be selected. These points were considered for the selection of the range of binding agents in this research:

1- To cover the mentioned range in the German leaflet M KRC [11] aiming to investigate the changes in material's characteristics in this range.
2- Considering the literature, a range to cover different kinds of material's behaviors say BSM (South African definition), hydraulically dominant and bitumen dominant. This gives the possibility of better tracing on the behavior change.
3- To have lower and upper limits of the bitumen and cement by roughly considering the economical points too.
4- Considering the number of the needed samples for the tests and the availability of the recourses (time and the testing equipment).

Considering the above-mentioned points, the range of 2.5% to 4.5 M.-% (dry weight of the granular mix plus cement, see Equation 3-3, section 3.4) was selected for bitumen and 1% to 3 M.-% (dry weight of the granular mix, see Equation 3-2 section 3.4) for cement. Then 5 different mix combinations were defined. First with a constant amount of cement equal to 1% and different amounts of bitumen and second fixing the amount of bitumen (3.5%) and changing the amounts of cement. The following notations will be used for referring these 5 mix combinations during this research: F2.5C1.0, F3.5C1.0, F4.5C1.0, F3.5C2.0, F3.5C3.0.

During the production of bitumen in most of the refineries, anti-foaming additives are applied to increase the efficiency of the production process which results in poor foamability of the bitumen. It is important to consider this when selecting the bitumen type. Nyfoam 60® which is a 50-70 penetration class of bitumen from Nynas bitumen company was selected for this research. It contains no anti-foaming additives and was delivered in 10-liter buckets(see appendix A for bitumen specifications). Portland cement type 1 (CEM I 42.5 N) was selected as the cement type for this research. It was stored in sealed buckets to be sure that moisture doesn't harm it.

3.3 Parent material (Mineral aggregates)

Parent material refers to the untreated material mixture. It contains the granular and the filler parts. As the research aims to consider the effect of binding agents, it was decided to fix the parameters of parent material in all mix combinations. As executed in the literature review, variations of the RAP characteristics may lead to different responses and behaviors of the resulting material. To omit this interaction effect, it was decided to use fresh aggregates for this research. Considering the literature, to have a compact gradation and a good bitumen dispersion, the Fuller method with n = 0.45 was selected (Figure 3-1). By having the maximum particle size (22.4mm), it is possible to calculate the percentage of mass passing from each sieve size.

$$P = \left[\frac{d}{D}\right]^n \times 100$$ **Equation 3-1**

d, selected sieve size (mm)

P, percentage by mass passing a sieve of size d (%)

D, maximum particle size (mm)

n, grading coefficient (0.45 selected)

Sieve size (mm)	Passing (%)
22.4	100
16	85.9
11.2	73.2
8	62.9
5.6	53.6
2	33.7
1	24.7
0.5	18.1
0.25	13.2
0.125	9.7
0.063	7.1

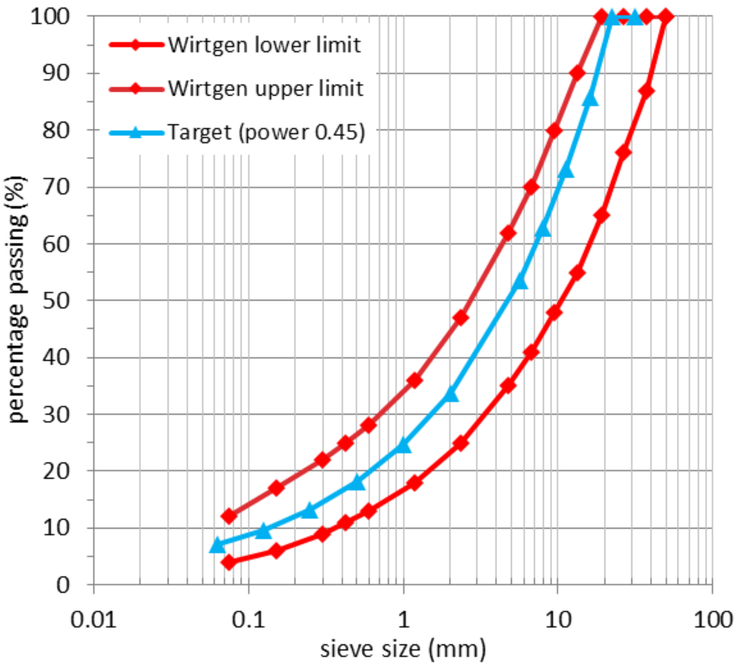


Figure 3-1: Target gradation table and curve, comparing with Wirtgen limits [7] for foamed bitumen mixes

To be sure of better behavior against the influence of the moisture from two aggregate type possibilities, Diabas was selected. It was delivered from an HMA plant near the city of Siegen in different fractions of 0.063 - 2, 2 - 5.6, 5.6 - 8, 8 - 11.2, 11.2 - 16 and 16 - 22.4 mm. After sieving each fraction to determine their exact gradation, the percentage of each fraction to reach the desired final gradation was determined by a try and error process with the help of an Excel® routine.

As the filler fraction is important in foam bitumen mixes, to have a constant and same quality for all mixes during the research, and as the fractions had almost no filler, it was decided to supply it separately and from another source type. Limestone powder (known as Kalkstein in Germany) is a common type of filler for HMA which is recommended and was applied in cold recycling projects too [17, 186]. It was selected as the filler type of this research. The important point was to keep the same gradation for all mix combinations and specimens.

3.4 Mixing Process and mixture production

Wirtgen WLB10S machine which is a laboratory scale foam bitumen producer was used to produce foamed bitumen from the Nyfoam 60. Based on the M KRC, the minimum amount of the expansion ratio (ER) should be 10 times and the half-life (HL) should be a minimum of 10 seconds. Different bitumen temperatures and foaming water amounts were tested to satisfy these minimum limits and based on the results, the foaming water amount was selected 3.3 M.-% of bitumen (12 L/h) and the bitumen temperature 180 °C (see appendix B for foaming parameters). The amounts of air and water pressures of the foaming machine were set to 5.5 and 6.5 bar, respectively. It is important to make the heating and circulation time of the bitumen in WLB10S as short as possible to avoid the aging of that. Based on the literature, keeping that under 8 hours is appropriate. For this research, the production was planned in a way that to keep this time around 6 hours. The amount of bitumen for the production of one batch is not too much, therefore the remaining bitumen from each bucket was reheated and reused normally more than one time. To control the acceptable numbers of bitumen reheating, the penetration index of the bitumen was determined after each round of using the bitumen and compared with the initial amount. Based on the results it is possible to reheat and use the bitumen up to 4 times [185]. As the penetration is not the perfect index, to be on the safe side, the production was planned in a way that each bucket of bitumen to be reheated only one time.

The aggregates were dried in the oven some days before the mixing date. Having the amount of needed mixture, the weight of aggregates' mix and then the amount of each aggregate fraction were calculated. The amount of cement was calculated based on its percentage and dry weight of the aggregates. The amount of bitumen (foam bitumen) was calculated based on its percentage and dry weight of aggregates plus cement. The same method was used for calculating the total water amount based on the selected percentage for water. The aggregate mix was prepared before the production and kept at room temperature to be sure that the temperature of the mix is almost the same for all the batches.

$$W_c = 0.01 * C * W_{agg.}$$ **Equation 3-2**

$$W_b = 0.01 * B * (W_{agg.} + W_c)$$ **Equation 3-3**

$$W_w = 0.01 * W * (W_{agg.} + W_c)$$ **Equation 3-4**

$W_{agg.}$, dry weight of aggregate mix

W_c, weight of cement

W_w, total weight of water

W_b, weight of bitumen

C, cement (%)

B, bitumen (%)

W, water (%)

Based on the literature review, the optimum moisture content (OMC) of the mix of aggregates and cement was selected as the amount of compaction moisture of the mix. To determine that, the moisture-density tests were performed on the mixture of the aggregates with 2% cement. The modified Proctor method was used for the compaction of the samples at different moisture contents (DIN 18127:2012-09). During the pre-tests it was concluded that by calculating the amount of water based on the dry weight of aggregates plus cement, the OMC doesn't change for cement percentages from 1 to 3%. Based on the moisture-density tests the amount of OMC was selected as 5.5% (see appendix C for the tests). Considering the literature findings and the amount of mixes filler (section 2.2.4), 75% of the OMC was selected as the mixing moisture content and the OMC as the compacting moisture content of the mixes respectively. As the aggregates were dried before, no moisture adjustment was needed.

It is important that the laboratory mixing process be similar to the real production process. For this research, a twin-shaft mixer was utilized (WLM30 from Wirtgen) which simulates the real field in-plant mixing. It has a maximum capacity of 30 kg for each batch. This mixer has the possibility of being connected to the foamed bitumen production machine so, the foam can directly discharge into the mixer. The mixing process started with dry mixing of the aggregate mix and cement, then adding the calculated amount of water to reach the mixing moisture content (75% OMC) and mixing for 30 to 60 seconds to get a homogenous mixture. Before injecting the foam, the mixer should work for a minimum of 10 seconds and continue to mix for extra 30 seconds after the foamed bitumen injection. After that, the rest of the water to increase the moisture content to OMC level was added and mixed for 60 seconds.

3.5 Compaction

To select the compaction method for this research, different points were considered together. As the existing common method for preparing HMA samples in Germany is to make slabs with the rolling compactor machine, and a research project on cold recycled mixes with emulsion bitumen used the same method [17], first this method was tried for a sample mixture. Because of the below reasons, it was not the appropriate method for this project:

1- The existing slab compactor was made for HMA and had an electrical heating system under the mold which can be problematic as the FCSM fresh mix has water.
2- Considering the number of mix combinations and the number of needed samples for different tests of each mix combination, and that from each slab only one sample may be extracted in practice, it needs a huge amount of raw material.
3- Considering special framework conditions for producing, curing and testing FCSM specimens, the very low rate of production of this compaction method didn't match the requirements.

4- The samples should be cored out of the slabs. Normal coring with water cooling may damage the samples especially the ones with lower amounts of binding agents. It also changes the moisture content of the specimens and needs extra time for drying.

The next option was the static compaction method proposed by the M KRC [11]. There were some points about this method too:

1- The amount of static load (49 kN) was not enough to reach an acceptable range of air voids content for the compacted FCSM mixes of this research. Even by decreasing the height of samples from 12.5 to 6 cm, the issue wasn't solved. Looking into the literature, (section 2.4.1) shows that the German static compaction has the lowest pressure compared to the other countries.
2- The compaction process is very much dependent on the amount of free water in the fresh mix and seems to work efficiently for mixes with high amounts of bitumen and especially emulsion bitumen (which was the base material for the M KRC).
3- Same as the rolling slab compactor the static compaction method had a low rate of production which was not appropriate for this research.

According to the literature review, Marshall compaction is the most common method which has the most advantages of high production rate. At the time of this research, the vibratory hammer was still under development and control tests and there was not a commercial version available on market. To study the static compaction method more, research was planned (as a bachelor thesis [185]) with the main goal to assess different compaction methods. Static load method (M KRC method) and Marshall (50 blows each side) were the two selected methods and modified Proctor compaction was used as the reference. Different mixes were produced with different percentages of foamed bitumen (3, 3.5, 4 and 5%) and 2% cement to investigate the effect of bitumen content on the volumetric characteristics of compacted specimens. The research had the second aim to validate the measuring methods for density, maximum density and the air voids content.

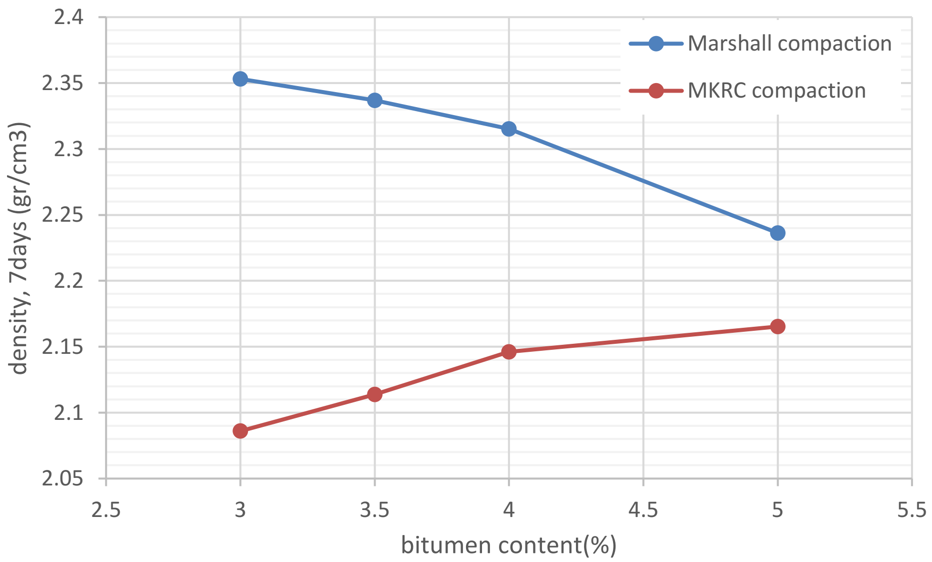


Figure 3-2: The effect of bitumen content on the density of the compacted specimens [185]

Figure 3-2 and Figure 3-3 are from the mentioned investigation (more results can be seen in the reference [185]).

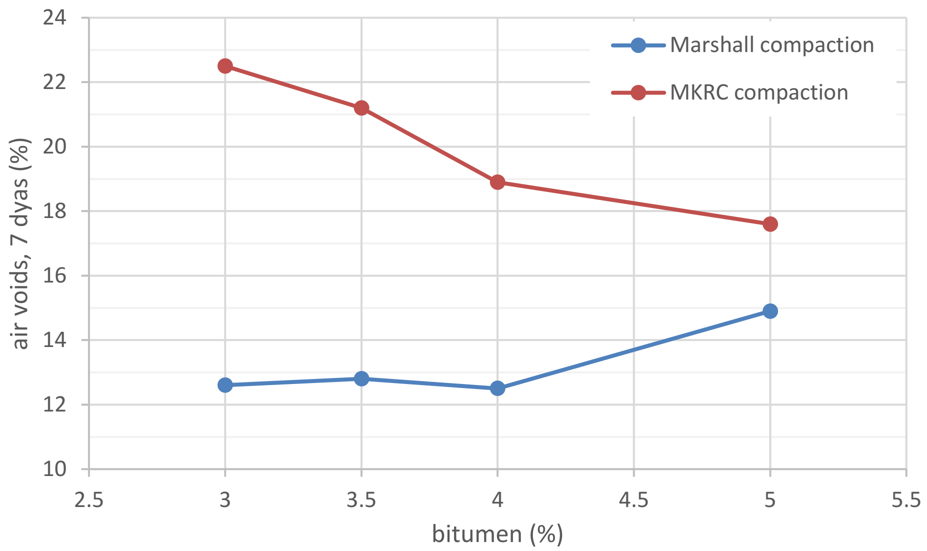


Figure 3-3: The effect of bitumen content on air voids content of compacted specimens [185]

It can be seen that the static compaction method of the M KRC results in lower density and higher voids content compared to Marshall compaction. By increasing the bitumen content, the density increases and the voids content decreases but still it doesn't reach the common ranges in the field (12 to 18%) and also the limit of less than 15% from M KRC.

Based on the results of the mentioned research, the number of needed samples from each mix combination and the time limit between mixing and the end of compaction, the Marshall compaction method (with 75 blows per face of the samples) with standard sample size, was selected for the main tests of this research. To have the same compaction conditions for all mix combinations, the amount of fresh mix for each specimen production was kept constant to 1200 gr. The amount was determined through the preliminary sample productions to get the height of 60 ± 2mm.

3.6 Curing

As explained already in the literature review (section 2.4.2), FCSM gains strength over time by the formation of bituminous and cemented bonds. During the curing period, water will drain out of the mix and these bonds will take its place. It is important to perform the laboratory tests at an appropriate cured state of material which reflects the long-term cured state in the field. There are different laboratory curing procedures in the literature (section 2.4.2). To select the main curing procedure for this research, two points were considered first the procedure should be the same for all different mix combinations and as there are mixtures with relatively high amounts of cement (3%) the procedure should be appropriate to be sure that it doesn't have a negative effect on the hydration of cement. The existing German procedure for curing from the M KRC code is the appropriate method for higher rates of cement in the mix and was adopted for this research as the main method. Based on this method, after compaction, and unmolding of the specimens (normally letting them be in mold or same situation overnight), the first two days, they should be cured with 20 ± 2 °C temperature and relative moisture of more than 95%. After that based on the selected age for the tests, curing will continue with room conditions (20 ± 2 °C temperature and relative moisture of 40 to 70%). It was decided to perform the tests at the ultimate cured state of the samples. The required time to reach this state (which the change in the mass with time is almost negligible) is different depending on the sample size, mix components and the climatic parameters of the samples' storage place (temperature and relative moisture), but it never reaches to zero [87]. As 28 days is the common standard duration for the mixes containing cement (2 and especially 3% cement), 28 days was selected as the curing duration of all mix combinations. So, after the first 2 days of moisture curing, they cured for 26 days in room conditions. This method will be referred to as *standard or M KRC curing* in the rest of the research. The tests of each group were planned in a way to start after this duration (see section 3.8). To assess the effect of curing time and temperature on material characteristics, besides the standard curing, a fast curing procedure was applied on additional specimens (with the F3.5C1.0 mix combination). Based on the findings from the literature the most appropriate method is the one from South African researchers by putting the specimens for 72 hours at 40°C. It was selected as the fast curing method for this research and will be referred to as *fast curing* in the rest of the text.

3.7 Volumetric indexes

Volumetric indexes are the physical characteristics of the material. for this research, 3 indexes, density, Maximum density and the air voids were selected. Considering that the material contains water at the time of compaction and its content will be decreased during the time (mainly because of evaporation and also the cement hydration), therefore its volumetric indexes are time-dependent too. To address this issue, these parameters should be measured at specific ages of the material/specimens. It is also important that density and maximum density be determined at the same age or curing state of the material. It should be noticed that there is not a dry state existing for this material in the field. The calculated moisture content of the samples after normal curing also shows that too (the amount is low and around 0.5%). There is a cured state in which the water evaporation reaches almost zero and the age of the sample is enough considering the cement hydration process (28 days). This can be taken as the dry state in the laboratory. Having the same state is important for performing the tests and determining the material's mechanical and performance characteristics.

The second problem arising from the existence of water and cement is that it is not possible to immerse the specimens in water for measuring the volume and determining the density because during this process the moisture content of the samples will change and it needs time to be sure that they reach to the dry state. For F3.5C1.0 mixes this time was minimum extra 7 days. Furthermore, this water and the needed waiting time after (to be sure they are dry again), may affect the mechanical characteristics because of the existence of cement. It was decided to determine the volume with the mathematical method by measuring the dimensions of the specimens which is also proposed in M KRC and TG2 (2009) guidelines too. The height of each sample was measured at 4 points (each with 90° apart) and the average was used. The diameter was measured on the middle height of the sample over two axis with 90° angles to each other. Maximum density (the density of the material in zero air voids state) can be determined theoretically with a mathematical method by having the amounts and densities of the mixes' components with the assumption that the cement particles and cement mortar have the same volume. It is also possible to use the existing test method known as the Rice method which is common for granular aggregates and HMA too (DIN 12697-5). These solutions were used, assessed and validated by some preliminary tests and then through the mentioned bachelor thesis [185]. Based on the findings, determining the maximum density with the test is more reliable than the theoretical calculation of that. The acceptance criteria is that the difference between the maximum and minimum of the 2 test results shouldn't be more than 0.025 gr/cm^3. When the difference is higher, then the third sample should be tested. It is recommended to prepare and test four samples.

3.8 Mechanical and performance tests

Considering the main inputs for the analytical structural design approach in Germany which are the material's stiffness, and its fatigue response function, therefore stiffness and fatigue were chosen as the mechanical and performance characteristics to assess the effect of binding agents amount. Considering the testing possibilities that existed for this research and the findings from the literature, diametrical loading mode on cylindrical samples was selected to determine and assess the FCSM's mentioned characteristics. As already mentioned in the literature this test method is not the perfect method but by considering all aspects of specimen production, preparing, testing and evaluating, it is the most appropriate existing one. Besides the stiffness and fatigue tests, static indirect tensile tests (see section 3.8.1) were performed too. A servo-hydraulic universal testing machine from Wille® company, type UL 63DYN/S2 was utilized for performing all the tests.

The criteria for the selection of the samples of each mix combination from each produced batch was the density of the samples calculated based on the method mentioned before. The difference between the maximum and minimum should be not more than 0.03 gr/cm^3.

3.8.1 Indirect Tensile Strength Tests (ITS)

The main aim of this test was to use its results as helping data to decide on the range of applying stress for cyclic stiffness tests. The applied force versus horizontal deformation graph can be used as a guide to decide on the linear elastic range. The ITS amount which is one of the results of the test can provide an overview of the amount of stress for cyclic tests. As the cyclic tests were planned at different temperatures (-10, 0, 10 and 20 °C), these tests should be done at the same temperatures too. The second aim was to evaluate the effect of binding agents by analyzing the vertical force vs. horizontal deformation data (strain at failure). The tests were performed according to DIN EN 12697-23, with a loading rate of 50 mm/Min. Samples were temperature conditioned for a minimum of 4 hours (for lower temperature 6 hours) for each testing temperature.

3.8.2 Indirect Tensile Cyclic Stiffness Tests

As the FCSM is a mix of two binding agents which one of them causes some level of viscous behavior in the material, it was decided to perform the stiffness tests at different temperatures and frequencies to evaluate the effect of cement and bitumen on the level of material's viscose response. The results will be used to construct the master curves of each mix combination which is an input for the structural design process. As there is no testing instruction or standard available in Germany (and EU) specifically for this group of materials, the working instruction to determine the stiffness and fatigue properties of hot mix asphalt samples with indirect tensile loading (AL Sp-Asphalt 09 [107]) were used as the base for the tests setups, conditions and analyzing of the results. It is very similar to DIN EN12697-26: 2012-06 (annex F). Tests were performed at different temperatures of -10, 0, 10 and 20 °C and different frequencies of 10, 5, 1 and 0.1 Hz. For each temperature, 3 samples were tested for all frequencies. The main adjustment done for the tests was that it was decided to perform them at a lower strain range than the mentioned one in the two above codes (which is 0.05 to 0.1‰). The reasons for that adjustment were: first, as the aim was to assess the viscous behavior of the mixes, therefore it was important to be sure that they remain in linear range behavior. The results of some pre-tests showed that when the norm proposed range (0.05 to 0.1‰) is used for the stiffness tests, the amount of damage in the specimens is higher than the acceptable limit (considering the decrease of the stiffness in the control test). Second, as the specimens were planned to be used for fatigue tests after the stiffness tests, it was important to be sure that they are not damaged. The decision is also supported by the findings from the literature too (see section 2.3.3). Each test was started with a low amount of stress (around 15% of the ITS of the mix at that temperature) then the resulted horizontal deformation was controlled and if it was in the safe range (determined from the load versus horizontal deformation form ITS results in each temperature), the load was increased till reaching to the limit. The other controlling criteria was comparing the amount of stress level with the ITS amount at the same temperature (not to go beyond 40% of ITS). At the end of each round of the stiffness tests of a specimen (at one specific temperature with different frequencies), a control test was performed at 10 Hz to evaluate the state of the sample (the difference should be less than 15%).

After the main tests, some extra tests were planned and performed aiming to address some open questions like the effect of curing, the effect of the selected horizontal strain range, the effect of specimen inhomogeneity. A series of specimens with the same gradation and mix design of F3.5C1.0 were produced with the hot mix asphalt production method and their stiffness were determined based on AL Sp-Asphalt 09 [107] as a reference to compare hot and cold production together.

It is important to mention that in the meantime the AL Sp-Asphalt 09 has been replaced with a new code (named TP Asphalt-StB Teil 26-2018) [187]. The main difference between these two is the approaches that they use for constructing the stiffness master curves which for FCSMs the old method had no problem. It is discussed more in chapter 4.

3.8.3 Indirect Tensile Fatigue Tests

Considering the selected range of binding agents for this research and the findings from literature (see section 2.5) it seems that fatigue can be the primary failure mode in this range (maybe not for the mixes with 2.5% bitumen). So, fatigue was selected as the performance index to evaluate if it is the relevant failure mode for the FCSMs. The second reason was that fatigue failure is the main and most common criteria in dimensioning pavement structures and the fatigue equation is one of the main inputs for analytical structural design.

Again, German code, AL Sp-Asphalt 09 [107] and DIN EN 12697-24: 2012-08 (annex E) were the bases for performing and analyzing the results for this test. No pretest was performed to determine the stress level for the tests. Because of sample damage in the range of pretest strains, then more than one sample was needed. The stress level of the first test of each round was chosen based on the results of the stiffness tests and the ITS amount. After having the results of the initial strain and the stress from each test, the amounts for the next tests were determined aiming to cover a wide range of initial strains too. Based on the DIN EN 12697-24: 2012-08 (annex E), it is acceptable to use standard Marshall samples size (100 ± 3 mm diameter and 60 mm height) for the mixes up to 22.4 mm maximum aggregate size. So, the Marshall compacted sample size was acceptable for fatigue. Based on the AL Sp-Asphalt 09, load type was cyclic sinusoidal with 10 Hz frequency, the tests first planned to be performed at 20 °C, after testing the first mix (F2.5C1.0), it was decided to decrease the test temperature to 5 °C for the rest of the samples (based on the results and also the literature too). Initial stiffness and strain were calculated from the data of cycles 98 to 102. For a round of fatigue tests to be able to extract the fatigue equation in the case of the HMA, normally 9 specimens are needed. To assess the effect of temperature, after main tests, some extra tests were planned at different temperatures (0, 10, 20 °C) on F3.5C1.0 samples. To assess the effect of curing and also the variation of the fatigue test results some extra tests were performed too. To be able to compare the effect of the hot and cold process of production together, the mentioned hot-made samples (samples with the same mix design of F3.5C1.0) for stiffness comparison, were used for fatigue tests too.

It is important to mention that in the meantime the AL Sp-Asphalt 09 has been replaced with a new code (named TP Asphalt-StB Teil 24-2018) [188] The main difference is the instruction that proposed to use the energy ratio method to determine the fatigue point. Based on the new code, a polynomic power 4 equation must be fitted to the data points in the range of ± 20% of the maximum point and the maximum from that will be the fatigue life to get macro cracks. For the results of this research, besides the maximum amount of energy ratio curve proposed by AL Sp-Asphalt 09, a polynomic power 2 equation was tested too.

3.9 Production and testing plan of the specimens

Planning the production and the tests of FCSM mixes is a challenging multi-criteria task as different requirements should be considered and be optimized. The requirements were:

1- To increase the homogeneity of the specimens for each set of the tests, it was planned to produce the needed samples for each set of the tests together in one batch.

2- To decrease the negative effects of the delayed time between the compaction of the first and last sample, which is not only the issue of moisture loss but also the issue of cementitious bonds, especially for mix combinations with higher than 1% cement, the compaction duration should be limited. Based on the pretests, it was found that up to 1.5 hours is acceptable, but the mixture should be kept covered. Considering the production rate of the Marshall compaction method (which is relatively high compared to the other methods), the number of specimens from each batch will be limited.
3- As mentioned before the heating and circulation time of the bitumen should be limited to avoid the negative effects of aging (Maximum 6 hours selected). the second point is that as the reheating and reuse of the bitumen should be limited too (for this research one time of reheating was selected), then it is better to produce as much as possible samples from each batch and each bitumen heating.
4- The tests should be performed after the curing time. Considering the selected mean age of 28 days, it is important to plan the production of the samples of different mix combinations after each other to minimize the waiting time (and also the total needed time) of the dynamic testing machine (as there was the only one available). The second point is that to decrease the effect of different ages of the samples because of the test's duration, the tests should be planned to be performed as compact as possible. It was decided to finish each round of the ITS and cyclic stiffness tests (all temperature and frequencies) in 2 days (in 27 and 28 days after compaction).
5- Some test results were necessary for the start of the other tests and considering the age conditions, they should be finished within a specific time limit. Bulk density and air voids of the specimens were needed to control the production and to select the samples for the tests. To determine the air voids, the maximum density is needed, and both bulk and maximum density should be determined at the same age. To decide on the stress level for the indirect cyclic stiffness tests, for each temperature, the results of the ITS tests were needed. so, all the needed samples for each round of the tests (ITS, cyclic stiffness and cyclic fatigue), should be produced together and in one day.

By considering all the requirements, the available resources and capacities in the laboratory, a comprehensive production and testing plan was prepared and optimized for this research. Considering the compaction points, in each round 10 to 11 samples could be compacted. For ITS tests 2 samples could be used for each temperature (8 samples together). For each temperature in the cyclic stiffness test, 3 samples were needed as 4 temperature levels should be tested, in one temperature level the same samples should be used. as the capacity of production (considering the mentioned points and limited resources) was 2 batches per day, it was decided to use the same samples of cyclic stiffness tests for fatigue tests. For each round of the tests (ITS, cyclic stiffness and cyclic fatigue), two batches were produced. The amount of material for each batch was determined to be able to compact 10 to 11 Marshall samples and also to have enough loose mix for four maximum density tests (15 to 18 kg for each batch). after drying the aggregate fractions and preparing the mix based on the desired gradation, one round of specimens' production and testing was as follow:

Day 1 tasks are:

1- Early in the morning start of heating of the first bitumen bucket (5 hours before mixing)
2- Dosing the needed amount of cement and water based on the mix combination
3- Starting the WLB10S and preheating, filling with heated bitumen
4- Producing the first batch of the mix

5- Discharging the bitumen from WLB10S. The process should be finished in a maximum of 6 hours.
6- Starting the compaction of the Marshall specimens (10 to 11 samples). It should be finished in a maximum of one and half hours. Taking enough amount from the loose mixture for maximum density tests and store it in a tray in laboratory room conditions.
7- Start heating the second bucket of bitumen to 180 °C (considering the 5 hours condition before the mixing)
8- As the amounts of Marshall molds were limited, so for the second batch, the samples should be demolded to be able to use the molds again. Based on the experiences, 3 to 4 hours were enough to be able to demold them. After demolding the samples were kept in a covered box over the night to simulate the state of keeping them in the mold.
9- After demolding the samples of the first batch (morning batch), the production of the second batch (afternoon batch) starts. The rest is the same as the first batch. only the compacted samples will remain in the molds overnight.

If the production of the batches starts at 8 in the morning, it will be finished normally till 17 in the afternoon.

Day 2 tasks are:

1- Demolding the samples of the afternoon batch, marking all the samples.
2- Preparing the moisture cabin and putting the samples in the cabin for 2 days curing in 20 ± 2 °C and more than 95% relative moisture content.

Day 4: Samples will be taken out of the moisture cabin and be stored in laboratory conditions for the rest of the curing process.

Day 26 tasks are:

1- Determining the mass of the samples and marking them for dimensions measurements to determine the bulk density of them.
2- Performing 4 maximum density tests on loose dried mixture samples.
3- Determining the acceptable samples for the tests based on the bulk density criteria.
4- Putting the samples in the dynamic testing machine for tempering and the 2nd group in the temperature cabin.

Days 27 and 28 tasks are:

1- ITS and cyclic stiffness tests. For each temperature, first the ITS tests and then based on the results the cyclic stiffness tests at that temperature for the range of frequencies. To use the time efficiently, a temperature cabin was utilized in parallel for temperature conditioning the samples of the next temperature level. The testing of each round started normally at 7 in the morning and continued to 23 in the evening.
2- Putting the first sample for fatigue test in the machine and temperature conditioning to 5 °C for tomorrow.

Day 29: Performing the fatigue tests. It was tried to select the stress level and to order the samples in a way that made the machine work overnight too. The aim was to shorten the effect of tests duration as much as possible.

In the case of batches with fast curing method, on the 2nd day instead of moisture cabin, the samples were cured at 40 °C for 3 days. The same curing was also used for the loose mixture for maximum density determination. On the 5th day, they were taken out of the temperature cabin and cooled to room temperature. After weighting, their dimensions were measured to determine their bulk density. Maximum density tests were also done on the loose cured samples too. Selected samples were conditioned for the tests in the next days.

To use the gap between days 4 and 27 and shorten the waiting time of the dynamic testing machine, the next rounds were so planned that the samples of the other mix combinations to be ready when the fatigue tests (or the tests) of the previous round were finished. The production and testing were continued over weekends too.

For the whole research program (pretests, compaction evaluation and the main research phases) 350 samples were produced which 255 of them were for the main research phases. The author was himself engaged in the whole process of production of these specimens and testing them. Table 3-1, shows a summary of the batches and the type of tests performed on them.

Table 3-1: Summary of the batches and the type of tests on their specimens (B. Nr. Stands for the batch number; F and C show the amounts of foamed bitumen and cement in M.-%; S shows the standard and F shows the fast curing; batch 23 is produced like HMA to be as a reference)

B. Nr.	Mixing Date	F-C (%)	Samples amount	Curing	Tests
1	18.09.16	2.5-1	10	S	ITS at -10, 0, 10, 20 °C
2	18.09.16	2.5-1	10	S	Stiffness-temperature and Fatigue @ 20 °C
3	30.09.16	3.5-1	10	S	Stiffness-temperature and Fatigue @ 5 °C
4	30.09.16	3.5-1	10	S	ITS at -10, 0, 10, 20 °C and 5 °C extra
5	16.10.16	4.5-1	10	S	ITS at -10, 0, 10, 20 °C and 5 °C extra
6	16.10.16	4.5-1	10	S	Stiffness-temperature and Fatigue @ 5 °C
7	29.10.16	3.5-2	10	S	Stiffness-temperature and Fatigue @ 5 °C
8	29.10.16	3.5-2	10	S	ITS at -10, 0, 10, 20 °C
9	14.11.16	3.5-3	10	S	Stiffness-temperature and Fatigue @ 5 °C
10	14.11.16	3.5-3	10	S	ITS at -10, 0, 10, 20 °C and 5 °C extra
11	19.11.16	3.5-1	11	F	ITS at -10, 0, 10, 20 °C and 5 °C extra
12	19.12.16	3.5-1	11	F	Stiffness-temperature and Fatigue @ 5 °C
13	28.11.16	3.5-1	11	S	Fatigue @ 0, 10, 20 °C
14	28.11.16	3.5-1	11	S	Fatigue @ 0, 10, 20 °C
15	13.12.16	3.5-1	11	F	Fatigue @ 5 °C
16	13.12.16	3.5-1	11	F	ITS at -10, 0, 10, 20 °C, with new filler type
17	02.01.17	3.5-1	11	F	Multi-step, Multi-round & 4 direction stiffness
18	02.01.17	3.5-1	11	F	Fatigue @ 5 °C
19	15.07.17	2.5-1	11	S	Multi-step stiffness: 5 °C,10 Hz & ITS: 5 °C
20	15.07.17	2.5-1	11	S	Fatigue @ 5 °C
21	21.07.17	3.5-1	11	S	Multi-step stiffness: 5 °C,10 Hz & ITS: 5 °C
22	30.07.17	0-2	11	S	Multi-step stiffness: 20 °C,10 Hz & ITS: 20 °C
23	13.07.18	3.5-1	11	-	Stiffness-temperature and Fatigue @ 5 °C
24	06.11.18	3.5-1	12	S	Stiffness-temperature and Fatigue @ 20 °C

It is important to mention that the first 10 batches were the first plan of this project but later by the progress of the tests, the other batches were planned to get more information on some raised issues. Therefore, the granular fractions were taken more than one time from the asphalt plant and the mixing ratios were adjusted (from batch 19) based on the results of the new gradation tests accordingly to be sure that the desired gradation is always achieved.

All the batches were produced, and their specimens were prepared and tested in the road laboratory of Siegen University (ifs Siegen) except the last two batches which were produced and tested in the German Federal Highway Research Institute (BASt) with the same materials transported from ifs Siegen.

Chapter 4

4 Results and Discussions

4.1 Important points regarding sample preparation in the laboratory

This section is based on the observations and the findings from the production and preparation of the FCSM specimens in the laboratory. As one of the questions in this research was about “how to produce and prepare test specimens in the laboratory”, it was decided to mention the important points here which can be a good guide for the next research programs.

Before the start of samples production, the calibration of the foaming machine (in this research WLB10S) should be controlled always. The calibration process is to be sure that the machine injects the right amounts of bitumen. The second control is the amount of water injection for the foam bitumen production. These two parameters should be controlled and if needed, be adjusted before the start of each daily production.

The temperature of the granular mix affects the dispersion of the bitumen in the mix and thus the quality of the mix. It is important that all the mixes of a research program have the same temperature. This will be more important when the program is long, and the production may be performed in two different seasons.

As the twin shaft mixer (which is nowadays the normal mixer for these materials) has the possibility of selecting between different rotation speeds, therefor different mixing energies will be inserted on the mix. It is important to select the appropriate combination of the mixing speed and duration and try to keep it constant during the whole of the production plan. When dry aggregates in fractions are used, it is better to make a dry premix (with low rotation speed) to make the mixture homogeneous. This mixing concept can be repeated after the addition of cement. The speed of the mixer should be kept low during these two mixing steps. It is also better to select a longer mixing time after the addition of the first part of water (the amount that is needed to take the mix to the optimum mixing moisture state). It is also recommended to consider some extra time between the mixing of this water and the injection of the foam. This will let the mix and moisture reach a better equilibrium state. The same should be considered after the mixing of the second part of water (the amount which is needed to reach the OMC level) and before the start of the compaction.

As mentioned before in the literature review, at the time of injection of the foam, the optimum state is that the mix to be agitated to have as much as possible higher volume. The free space in the mixer housing will affect this state so it is recommended to plan the production process in a way that not to fill the mixer to its highest capacity and also to keep the amount relatively constant for all the batches if possible.

The amount of moisture in the mix at the time of foam injection (mixing moisture content), has a high effect on the form characteristics and distribution of the bitumen mastic droplets. This will later affect the resulting material's different characteristics. If the moisture content is high, it leads to a decrease in the amount of free fine particles available for bitumen droplets to stick to them. The result is then a bigger bitumen mastic phase and inappropriate distribution of that in the mix. This will increase the level of mixes inhomogeneity and also the variation of the tests' results. To decide on the optimum amount of mixing moisture, it is important to be sure that the moisture-density tests are done right and precisely.

It is important to emphasize the high effect of compaction on the results. By the advances in compaction technology and introduction of new rollers, the compaction energy in the field is higher than laboratory, which means in the field it is possible to compact the mixes with lower moisture levels. This should be kept in notice in the laboratory in deciding on the amount of mixing water and compaction water content. Using only the mixing moisture content may lead to inappropriate level of compaction in the laboratory and as in most of the cases the samples' air void content is not determined and controlled, then there will be no reference to notice that.

If the research requires the same parent material gradation for all the specimens, it is important to acquire enough material and to homogenize all together with an appropriate method. If different fractions are used to produce the desired blend, homogeneity and the gradation of 0.063 - 2 mm fraction is more important, it is recommended to use some intermediate sieves to get a better understanding of the gradation of this fraction. When the research plan needs to produce more samples and new material is needed, it is important to precisely determined the gradation of the new material and adjust the mixing ratios of each fraction to be sure that the desired gradation is still reachable.

The surface of the cured samples is covered by a layer of loose fines (dust) which can cause to inappropriate state for the horizontal deformation data collection of the sensors, especially during the cyclic stiffness tests. It is recommended to use grinding paper (which is used for cleaning the wood surfaces) and then air (low pressure) to clean the round surface.

Producing and preparing bigger sample sizes are recommended if the available material and production / testing equipment of the project allows doing so.

After the end of each test, it is important to check the temperature of the specimens to be sure that the desired temperature was achieved for the test. The temperature should be measured in the middle area of the broken faces. It is also recommended to take samples for determining the moisture content of the specimens. Knowing the moisture content is important especially when different curing methods are applied.

4.2 Volumetric parameters

As mentioned before (section 3.7), bulk density of the specimens, maximum density and the air voids content were selected as the volumetric parameters. Compared to other parameters, the maximum density is a reference that is not dependent on the compaction and reflects the density of the mixture in zero void content state. This parameter can then be used to determine the air voids content of the specimens which can be applied as an appropriate index for compaction evaluation and comparing different methods together.

Figure 4-1, Figure 4-2 and Figure 4-3 show the changes of each parameter versus the bitumen and cement contents. The first 10 batches' data were used for the graphs. The standard curing was utilized for all the samples. As for each mix combination two batches were produced (see section 3.9), both are presented in the graphs. For the maximum density, each column is the average of the accepted results of each set of the tests for each batch. for the bulk density and the voids content, each column is the average of the results of the whole samples of that batch.

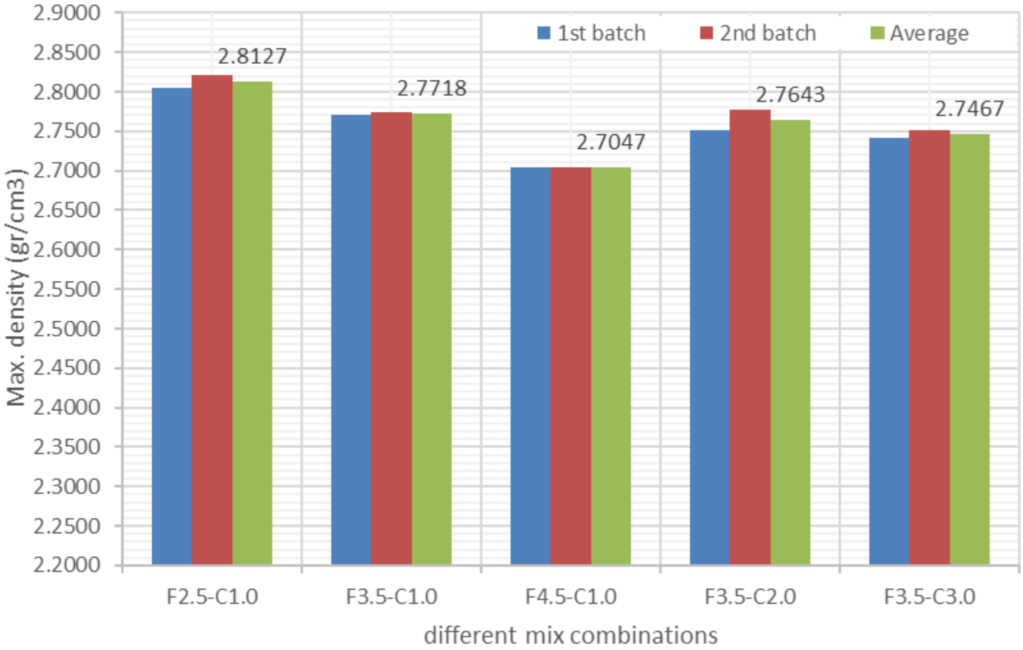


Figure 4-1: The effect of binding agents (bitumen and cement) on the maximum density

Considering Figure 4-1, it can be seen that increasing the bitumen causes the decrease of maximum density. This is because of the lower density of bitumen compared to the granular blend. The same trend can be seen for the cement too, but the rate is much lower than the bitumen and can be considered as constant. As the cement has a higher bulk density than the aggregates, if the mathematical method for determining the maximum density is applied, this would lead to an increase in maximum density by increasing the cement content. It is important to mention that the cement will not remain as cement and will hydrate and produce cemented products that are different in density than the cement particles. This is the short come of the mathematical method instead of performing the test to determine the maximum density.

Figure 4-2 shows by increasing the bitumen, the resulting bulk density of compacted samples decreases. It confirms the observed trend during the preliminary tests for Marshall compacted specimens (see section 3.5). Generally, the bulk density of the compacted specimens is the result of the interactions of the compaction method (type and energy) and the aggregates' characteristics (i.e. gradation and surface properties). Increasing the compaction energy decreases the voids and leads to a higher density. By adding the bitumen it will substitute with the air voids and lead to an increase in density. If the compacted skeleton has already a low amount of air voids (which depends on the compaction type and energy), then increasing the bitumen content will change the compacted skeleton of the aggregates. Depending on the new skeleton characteristics, the density will decrease but the resulted voids content may increase or decrease.

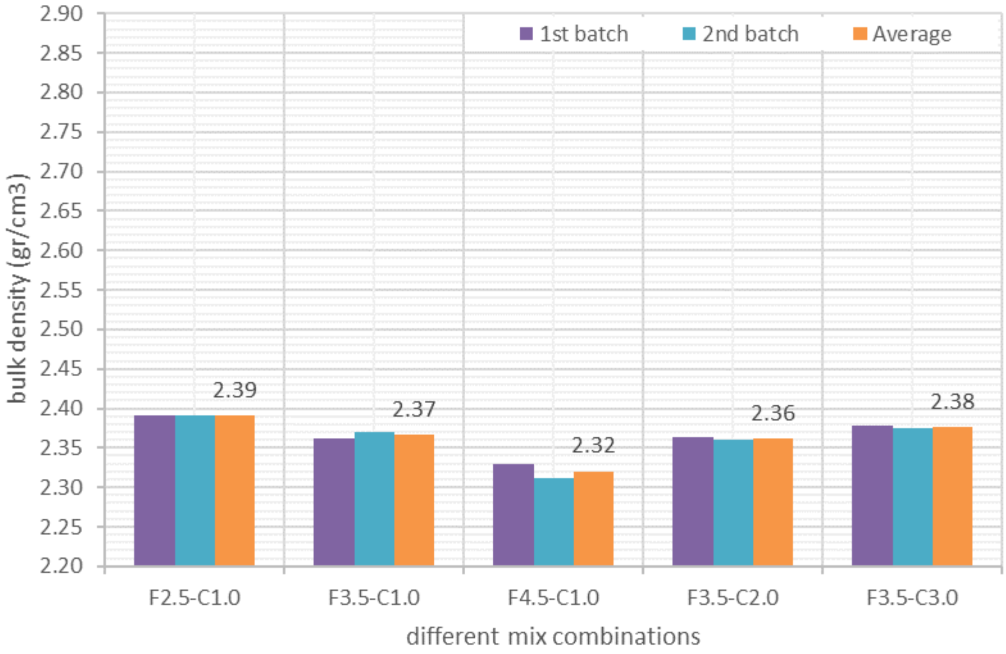


Figure 4-2: The effect of binding agents (bitumen and cement) on the bulk density

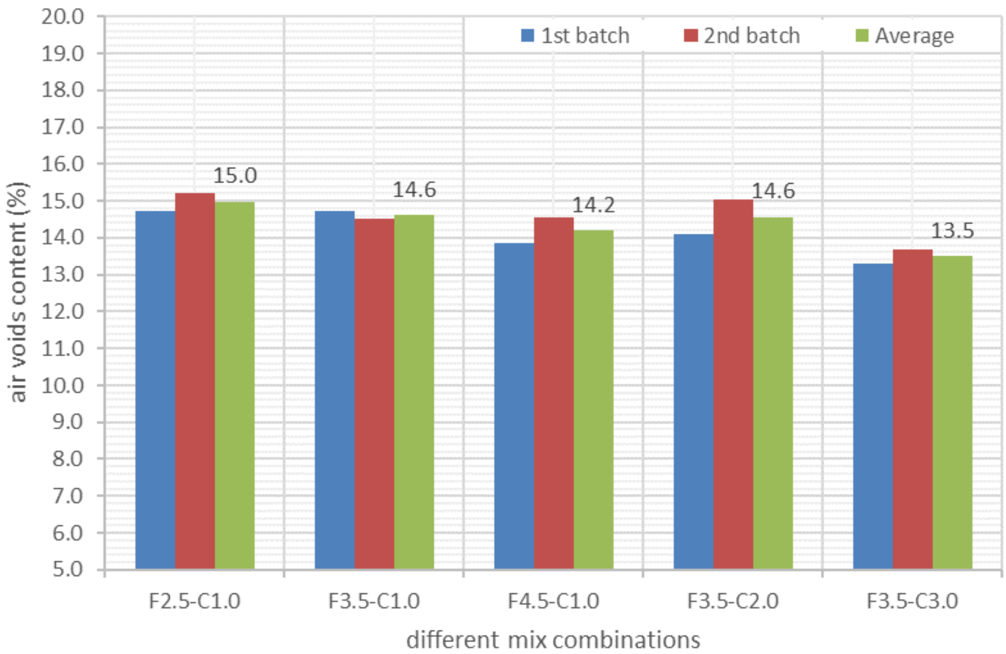


Figure 4-3: The effect of binding agents (bitumen and cement) on air voids content

As the bitumen doesn't coat all the aggregates in cold foam bitumen mixes, increasing the bitumen will affect its distribution and also the size of the mortar phase particles. These particles are vicious and when they become bigger, the impact-compaction type (Marshall) becomes less effective to be able to squeeze them into the coarse aggregates' skeleton. This can also lead to a decrease in the resulting density. But as the mortar phase has lower voids, that led to a decrease of the voids content (see Figure 4-3). Considering the cement effect, its behavior is different from bitumen. The bulk density of compacted specimens increased with cement content increase and the voids content decreased as a result of that (see Figure 4-3).

Based on M KRC [11] the voids content of the compacted mix should be between 8 and 15%, which all the mixes (all combinations of bitumen and cement) lay in this range. It shows that to achieve this range a high compaction effort is needed.

As mentioned before (see section 3.6) besides the standard curing method, for a selected mix design, a fast curing method was used to evaluate the effect of that. The curing method may affect the volumetric parameters of the mix mainly because of the difference in the remaining moisture after the end of the process. This type of difference can be tracked by comparing the volumetric parameters of the same mix combinations, cured with different methods. Table 4-1 and Table 4-2, show volumetric parameters of different batches of F3.5C1.0 made at different dates.

Table 4-1: Volumetric parameters of F3.5C1.0 different batches, cured with the standard method

Batch Nr.	Mixing date	Ave. of Max. Density (gr/cm^3)	Ave. of Bulk Density (gr/cm^3)	Ave. of Voids (%)
3	30.09.16	2.7700	2.36	14.7
4	30.09.16	2.7735	2.37	14.5
13	28.11.16	2.7405	2.33	14.8
14	28.11.16	2.7471	2.31	15.8
21	21.07.17	2.7282	2.35	13.6
24	06.11.18	2.7185	2.34	13.7
25	11.12.18	2.7190	2.29	15.6
Ave. Total		2.7424	2.34	14.7
STDEV		0.0210	0.03	0.79

The first series (Table 4-1) cured with standard and the second (Table 4-2), with fast curing method. Looking at the averages and standard deviations of the three volumetric parameters and comparing them together shows a very low (almost no in case of maximum and bulk densities) difference between them. This may prove that the two curing methods, may lead to the specimens with the same volumetric parameters in mixes with 1% cement content.

Table 4-2: Volumetric parameters of F3.5C1.0 different batches, cured with the fast method

Batch Nr.	Mixing date	Ave. of Max. Density (gr/cm^3)	Ave. of Bulk Density (gr/cm^3)	Ave. of Voids (%)
11	19.11.16	2.7186	2.34	14.3
12	19.12.16	2.7199	2.33	14.2
15	13.12.16	2.7115	2.33	14.9
17	2.01.17	2.7544	2.30	16.5
18	2.01.17	2.7560	2.28	17.5
Ave. Total		2.7321	2.32	15.5
STDEV		0.0191	0.02	1.31

The samples for maximum density in standard curing batches were left in room conditions for the whole curing duration of the specimens of those batches but in case of fast curing batches, they were put in the temperature cabin, like their specimens (72 hours at 40 °C). These same results (see the average and standard deviation of maximum density of total batches in two tables) show that both methods led to almost the same remaining level of moisture in the loose samples after the end of each curing method.

Considering the low amounts of standard deviations of the bulk densities and voids contents in both curing methods shows that the Marshall compaction method has a good repeatability level of specimen production. This can also be seen by comparing the densities of specimens of two different batches of F2.5C1.0 (batches 1 and 19) which were produced in two different times with almost one-year difference (18.09.16 and 15.07.17).

Cold mixes have higher voids contents compared to hot mixes. This is partly because of their lower compact ability and also the existence of water which after curing (evaporation) causes an increase in the voids' content of the specimens. Table 4-3, shows all the mixes with 3.5% bitumen and 1% cement. The last batch is the one made like hot mix asphalt. Comparing the bulk densities, all the cold batches have lower density and higher air voids' content than the hot made one. Looking at the standard deviation of the bulk density of samples from each of the cold-made batches and comparing them to the hot-made one, all of them (except one, Batch 13) are lower than the samples of the hot-made batch. This shows that the selected compaction method (Marshall compaction with 75 blows per side) was consistent in producing specimens with an acceptable range of deviations.

Table 4-3: The effect of production method (hot or cold) on specimen's air void content

Batch Nr.	Production Method	Mixing date	Ave. of Bulk Density (gr/cm^3)	Standard deviation	Ave. of air voids (%)
3	cold	30.09.16	2.36	0.008	14.7
4	cold	30.09.16	2.37	0.014	14.5
13	cold	28.11.16	2.33	0.020	14.8
14	cold	28.11.16	2.31	0.019	15.8
21	cold	21.07.17	2.35	0.017	13.6
24	cold	06.11.18	2.34	0.017	13.7
25	cold	11.12.18	2.29	0.016	15.6
11	cold	19.11.16	2.34	0.016	14.3
12	cold	19.12.16	2.33	0.013	14.2
15	cold	13.12.16	2.33	0.012	14.9
17	cold	2.01.17	2.30	0.013	16.5
18	cold	2.01.17	2.28	0.014	17.5
23	hot	13.07.18	2.44	0.020	9.66

4.3 Indirect Tensile Strength

The test was discussed already in chapter 2 (section 2.3.2). Normally the results of the ITS test are shown in a graph of vertical force vs. vertical deformation of the specimen till splitting point. Figure 4-4 shows the typical shape of this graph for two different mixes (at 20 °C) as an example. Looking at the graphs after the setting phase, vertical deformation increases with load till reaches the maximum point (failure point), then continuous downward till the end of the test based on the stopping criteria.

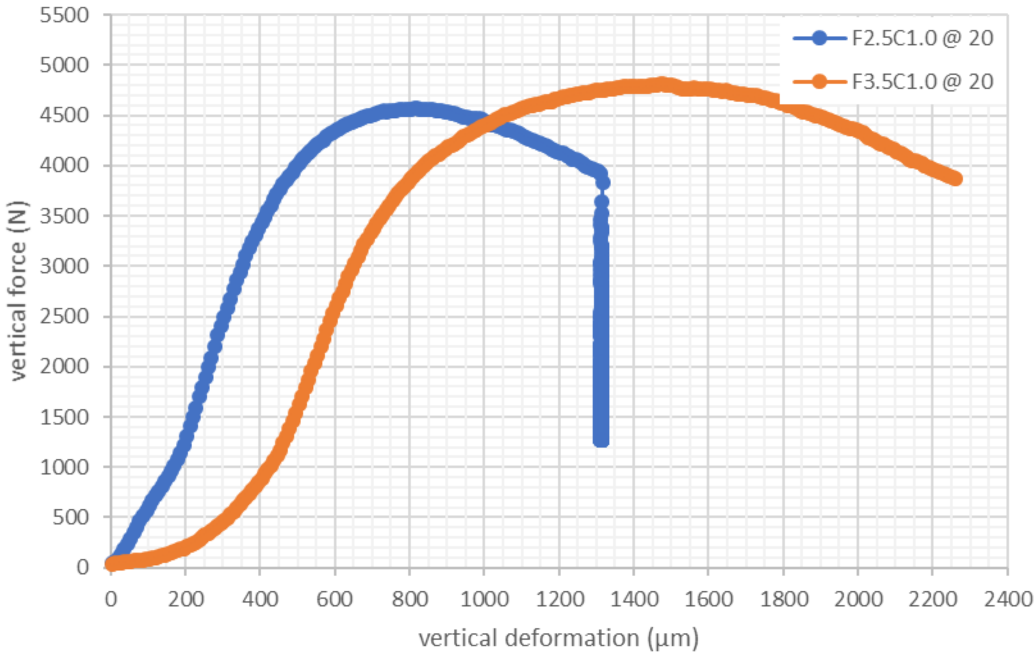


Figure 4-4: Typical shape of vertical force vs. vertical deformation in ITS tests

The idea of having this test on the FCSM specimens in this research was to get an impression of the horizontal deformation limit for the cyclic stiffness tests. Horizontal deformation was measured during the test too. The same frame and positioning setup (which is used for fixing the measurement sensors during the indirect tensile cyclic stiffness and fatigue tests) was applied, aiming to record the horizontal deformation of the specimens during these tests too. Two stopping criteria were defined for the testing machine to be sure that after reaching the failure point, the loading stamp doesn’t damage the frame and the sensors. The first one was to stop the test when the load reaches half of the maximum recorded amount and the second was a displacement criterion based on the free space between the frame and the supporting system under that. It is important to mention that the measurement range is limited to the free space between the specimen and the fixing frame and the sensors’ capacity. There are also some arguments that the clamps may affect the results of the test. As the important part of the test was to have the data till the maximum load (failure point) point, the limiting problem and the mentioned concern didn’t affect the measured data.

Figure 4-5 shows the typical shape of vertical force vs. horizontal deformation graphs during the ITS tests. As can be seen in the figure the shape of the graph after the failure point may be different. The specimen with higher bitumen content (F3.5C1.0), deformed more and therefore caused the sooner stop of the test compared to the one with a lower bitumen amount. This led to not catching the end sharp drop in this case.

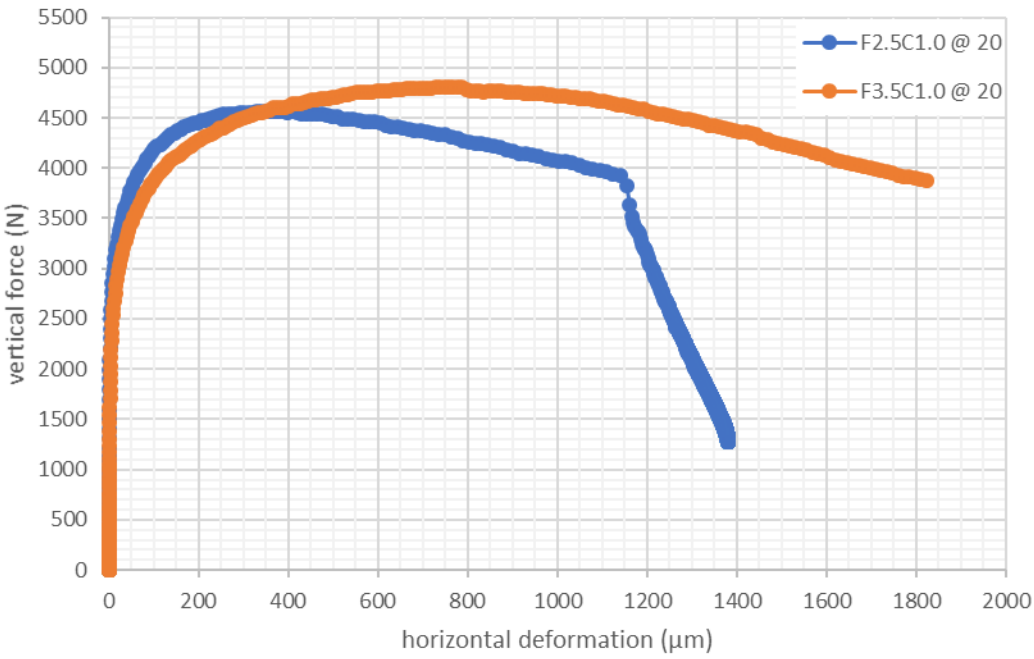


Figure 4-5: Typical shape of vertical force vs. horizontal deformation in ITS tests

In this test, the load is inserted on the sample with a constant increase of vertical deformation which is relatively high (50 mm/Min.) compared to the normal amount for cemented materials. In a composite material like FCSM first, the weak bonds (mostly formed from free filler particles and water) break, then the rigid bonds (more cement dominant) will break and later the flexible ones (more bituminous bonds). The test's temperature may affect the last one too. As the aim of cyclic stiffness sweep tests were to investigate the effect of binding agents on the response of FCSMs at different temperatures and frequencies, it is logical to perform the tests in an appropriate range of horizontal strain which both of the main bond types can be assumed to remain reasonably intact. Initiation of micro damages can be traced from the graph of vertical load vs. horizontal deformation. This limit can be used as a guide in cyclic stiffness tests to keep the specimens at an acceptable level of microdamage. Figure 4-6 depicts the first part of the horizontal deformation graph from the F3.5C1.0 specimen in Figure 4-5. Two points are highlighted on the graph. One of them shows the point with 45% of maximum force (2164.8 N, 5.3 µm) which is the suggested point in M KRC to determine the elastic modulus but the second one is the approximate point which can be considered as the end of the linear part of the graph (1.2 µm horizontal deformation in this example). An approximate amount for this point was determined from the ITS tests for each temperature and mix combination before the start of the cyclic indirect tensile stiffness tests of that mix combination. The results agree with the limit of 2 µm horizontal deformation selected by some researchers (see section 2.3.3). Considering the amount of horizontal deformation versus vertical load, in almost all the mix combinations (except some temperatures in F4.5C1.0), when the load is at 30% of maximum load (failure load), the amount of horizontal deformation is around 2 µm or lower. As the vertical load is in direct relation to the amount of ITS, this finding shows that in this range of binding agents 30% of ITS at each temperature, can be a good estimation for selecting the limit of stress level for cyclic stiffness tests on that temperature. This confirms the assumption used by Kavussi & Modaressi [102] (see section 2.3.3). It also shows that for the range of mixes in this research the 45% of maximum load (which is recommended by M KRC) will cause micro damages in the specimens. These findings were used as a guide to control the level of applied stress for each cyclic stiffness test.

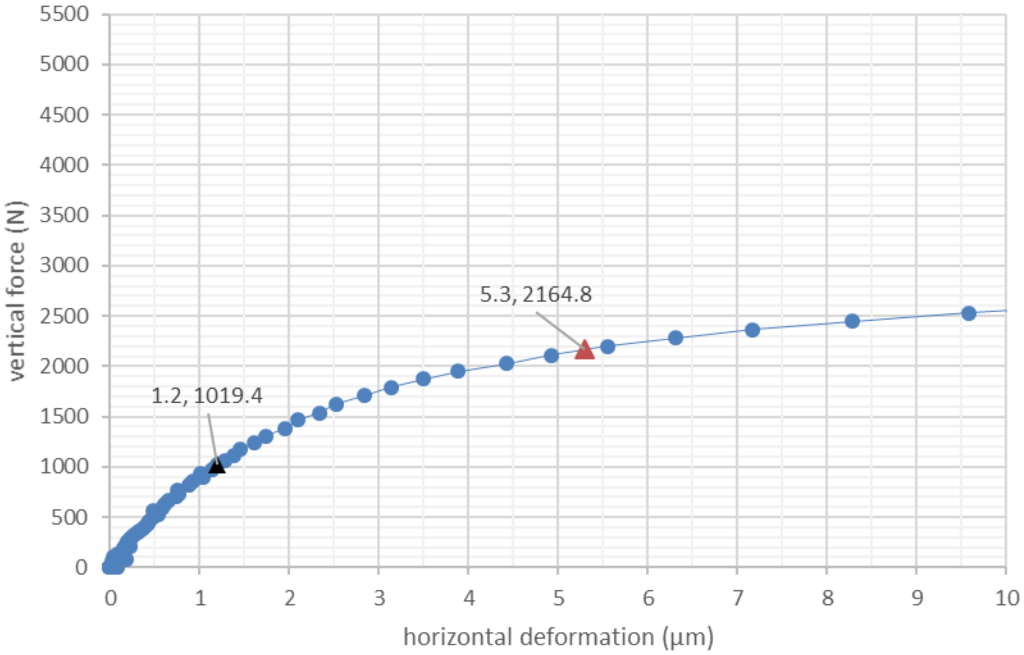


Figure 4-6: The first part of the horizontal deformation graph from the ITS test

Maximum force and the related amount of horizontal deformation (force and deformation at the failure point) can reflect the characteristics of the bond phases (cemented filler bonds and bituminous mastic) in this composite material. It can be seen from the deformation graphs that the deformation at failure point changes with mix type and temperature, so they can be used as indexes to assess the composite effect of binding agents' content at different temperatures. This will be discussed later in section 4.5.

As mentioned before, the main aim of ITS tests for this research was to get an insight into the limitation of horizontal deformation for cyclic stiffness tests. So, the number of repetitions and the range for the test temperatures were selected according to that. Later, it was decided to use them for some extra assessments on the material's characteristics. Part of the results were reported and assessed in the second Bachelor thesis done as a part of this research [189]. After the end of the first phase of the tests and after assessing the ITS results, it was figured out that the variation between the results of two test samples at some temperatures and mix combinations was higher than the others. Later for other purposes, some extra ITS tests were performed on mix combinations of F2.5C1.0 and F3.5C1.0 (batches 19 and 21 accordingly). The tests were done at 5 °C but with a higher number of samples for each mix combination (9 and 10 accordingly). Based on those results it is possible to determine the standard deviation of the tests' results. With the assumption that the standard deviation of the test results for each mix combination can be taken equally for all temperatures, then it is possible to determine the critical range although when the number of the samples at each temperature is only 2, for 95% level of reliability (FGSV 926/1 [190]). This assumption is acceptable as 5 °C is the middle temperature in the range from -10° to 20 °C. Based on the calculated critical range, it is possible to check if the results of the tests at each temperature have an acceptable difference or not. Table 4-4 and Table 4-5 show the results of ITS tests at

5 °C for F2.5C1.0 and F3.5C1.0 mixes. Considering the standard deviation amounts, they increase with the bitumen content.

Table 4-4: ITS test results of F2.5C1.0 mix at 5 °C

F2.5C1.0									
Sample ID	19-2	19-3	19-4	19-5	19-6	19-7	19-8	19-9	19-10
ITS (kPa)	712	759	626	720	663	686	633	684	706
Standard deviation	43								

Table 4-5: ITS test results of F3.5C1.0 mix at 5 °C

F3.5C1.0										
Sample ID	21-1	21-2	21-4	21-5	21-6	21-7	21-8	21-9	21-10	21-11
ITS (kPa)	896	875	824	929	804	875	784	749	773	794
Standard deviation		60								

Higher amounts of bitumen increase the size of the bitumen mastic phase droplets and also change their distribution. Bigger size affects the homogeneity of distribution and leads to an increased level of variation between the ITS test's results. Based on this observation and the logical argument, it is expected to have higher deviation amounts for the mixes with 4.5% bitumen. Assuming that the increase is linear, the standard deviation for F4.5C1.0 can be determined based on the other two amounts (equal to 76). Based on FGSV 926/1 leaflet [190], the critical range between the two results by considering a level of reliability equal to 95%, can be up to 2.77 of the standard deviation.

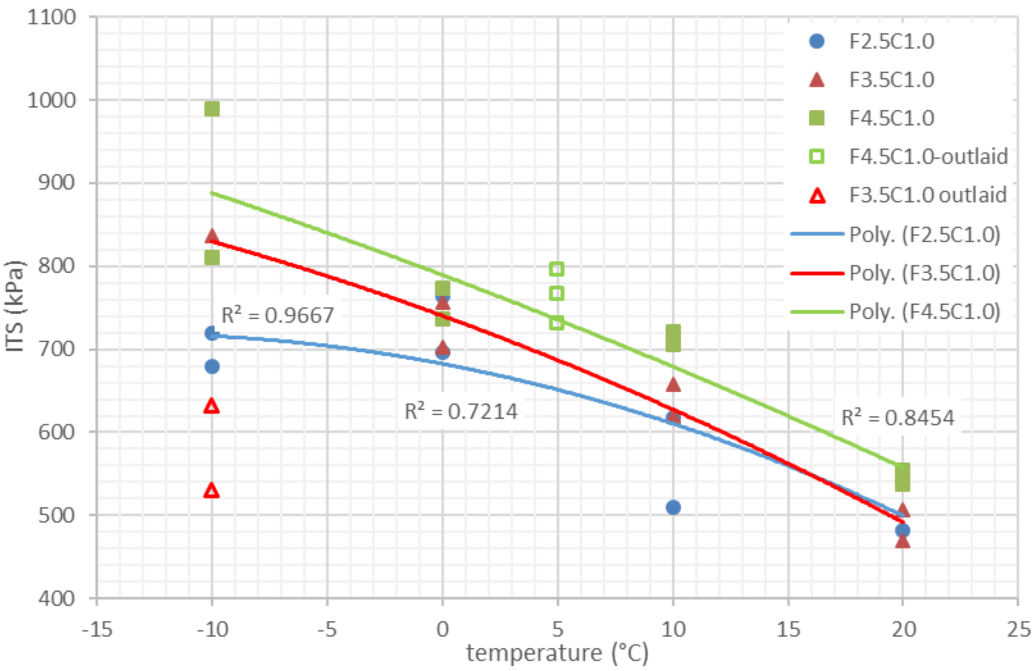


Figure 4-7: The effect of bitumen on ITS amounts at different temperatures

Figure 4-7 shows the ITS results at different temperatures and bitumen contents. Based on the mentioned control method, the data points of each temperature seem to be acceptable. As the results of F3.5C1.0 at -10 °C were susceptible, an extra sample (from the later made batch Nr. 21) was tested to control the results at that temperature and was used for the fitting equation. Referring to the graphs, the general trend is that the ITS increases with a decrease in temperature which confirms the temperature-dependency of FCSM as the bitumen mastic phase becomes stiffer at lower temperatures. Temperature dependency is lower for F2.5C1.0 mixes which is logical because they contain a lower amount of bitumen.

Looking at the -10 °C test results, the variation between the results is higher than in other temperatures. As the moisture in the samples will never reach zero (the moisture in specimen 1-10 was 0.73% at the time of testing at -10 °C), it may freeze at lower temperatures (-10° and 0 °C) and as it is randomly distributed in the specimen, it affects the results and can be one of the scatter sources between them.

The 3 outlaid points at 5 °C (the F4.5C1.0 graph) are the samples that are based on the density criteria (the difference between the maximum and minimum less than 0.03 gr/cm^3) were not accepted. Their densities were 2.29, 2.35 and 2.36 gr/cm^3. Looking at the graph, their results agree with the trend of other data points which shows that either density calculation method (based on a mathematical calculation of the samples' volume) may not reflect the real density or there are other influencing factors on the test results which can mask the density criteria in FCSM. It is recommended first to test all the samples and then check the density criteria.

Looking at different tensile tests' results on hot mix asphalt, tensile strength increases with the decrease of the temperature up to a pick point. The pick-point temperature is dependent on the bitumen type and the testing mode (direct tensile, semicircular bending or indirect tensile) [191]. The test is normally done up to -20 °C and in normal mixes the pick-point lays between -5° to -15 °C but sometimes with modified bitumen mixes, lower temperatures are needed to catch the pick-point. The same trend can be seen in F2.5C1.0 too but for the other 2 mix combinations may be more test points are needed on the lower temperature side (up to -20 °C) to decrease the scatter and be able to catch the right behavior. As this behavior is because of the existence of bitumen so it can be generalized to FCSMs too. This behavior can be a helpful way in selecting the test outliers and reconstructing the graphs as the 2nd approach. Figure 4-8 is based on this thinking; looking at the graphs shows the temperature dependency of the mixes (same as the graphs of Figure 4-7) so, the main conclusion is still the same and didn't change with this approach too. Considering the fitted equation to the data, it is possible to determine the pick point temperature for each mix combination. For F4.5C1.0 and F3.5C1.0 mixes, it is -6.4 °C and for F2.5C1.0 the amount is -3.8 °C. keeping in mind that the amount of bitumen is lower in these mixes than in hot mix asphalt (normally asphalt wearing courses are used for this test) and also its distribution in the mix is different from hot mix asphalt samples, the results seems logical.

The same observation was reported by Dal Ben & Jenkins [103], they performed ITS at different temperatures ranging from 40° to -10 °C. Their investigation showed a deviation from linear relationship below 10 °C.

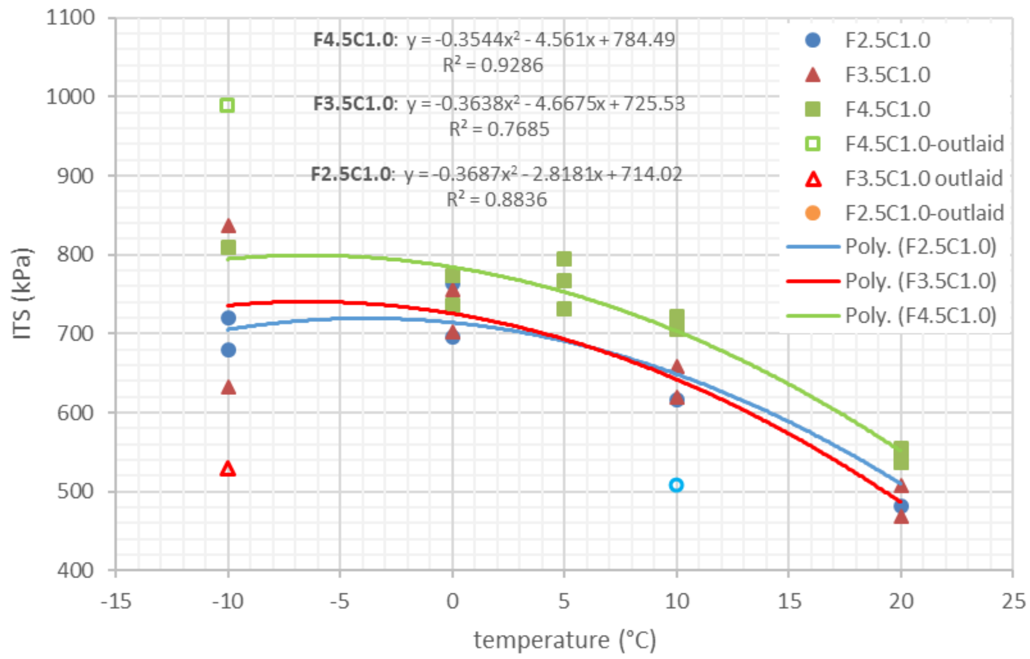


Figure 4-8: The effect of bitumen on ITS amounts at different temperatures, 2nd approach

It is also possible to assess the effect of cement content on the temperature dependency of ITS too. Figure 4-9 shows this effect for mixes with 3.5% bitumen and 1, 2 and 3% cement contents. It is possible to see that the ITS increases with increasing the cement content which is logical because increasing the cement affects the strength of cemented filler bonds and also affects the strength of the bituminous mastic too.

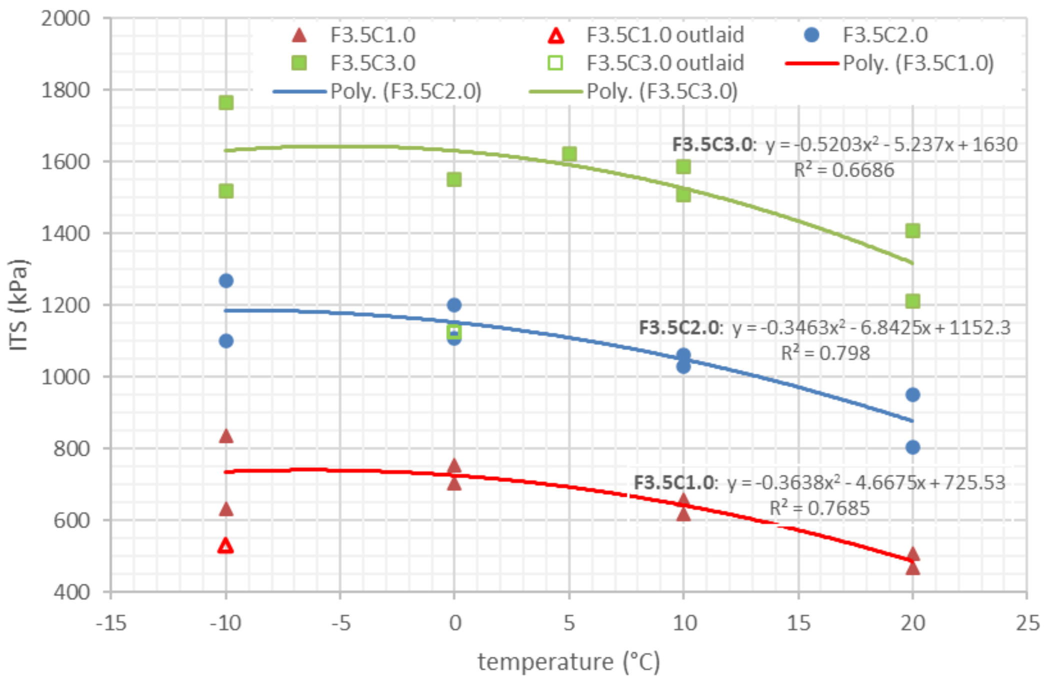


Figure 4-9: The effect of cement on ITS amounts at different temperatures

The first point by comparing Figure 4-8 and Figure 4-9 is that the effect of cement on the increase of ITS is more than the effect of bitumen on that. Comparing the graphs of Figure 4-9 together, shows that the ITS amounts are still temperature-dependent. It is not affected by the cement content. Considering the material's microstructure model (see section 2.1.2) it seems that the cement mostly contributed to the strengthening of the free filler phase. This is reflected in the higher ITS results by increasing its amount but the temperature dependency of the ITS results is affected by the bitumen mastic phase.

Considering the scatter of the F3.5C3.0 mix results at -10 °C and the fact that the addition of cement may shift the pick-point ITS to lower temperatures (this can be seen in the trends of F3.5C1.0 compared to F3.5C.20), it is possible to accept that the lower ITS of F3.5C3.0 mix at -10 °C is an outlier. Figure 4-10 is based on this approach. Pick point temperatures are -6.4 °C for 1% cement, -9.9 °C and -19.9 °C for 2 and 3% cement, respectively.

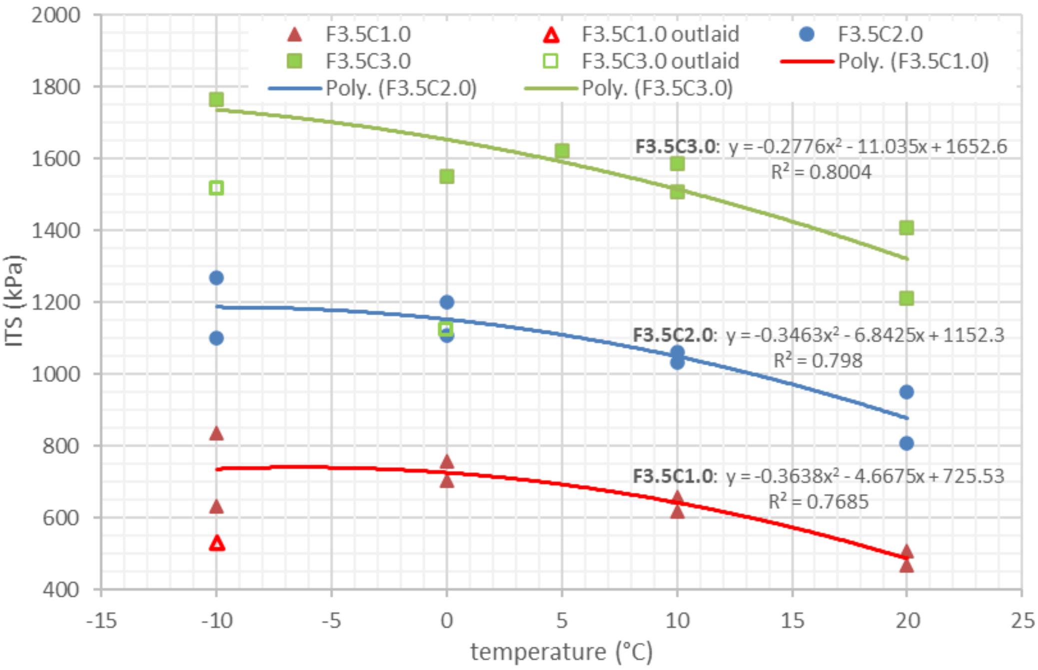


Figure 4-10: The effect of cement on ITS amounts at different temperatures, 2nd approach

Based on the Wirtgen recycling handbook [7], when the ITS at 25 °C, is more than 500 kPa, the mix may consider as hydraulically dominant. By extrapolating, the ITS results to 25 °C, for F3.5C1.0 the ITS will be 381 kPa, for F3.5C2.0, 765 kPa and for F3.5C3.0 is 1174 kPa. It shows that 2% of cement transferred the mix to the hydraulically dominant class. With an interpolation between the results of 1and 2% cement at 25 °C, the amount of cement for an ITS of 500 kPa is around 1.3%. This finding agrees with the general recommendation in the literature to keep the cement amount less than 1.5% if the aim is to control the rigidity of the resulting mix.

Based on M KRC [11] the ITS of the specimens after 28 days of curing, tested at 5 °C should be more than 750 kPa and less than 1200 kPa. Considering the presented results of different mixes (see Figure 4-8 and Figure 4-9), in the case of 3.5% bitumen, the amount of cement should be slightly more than 1% to be sure that the lower limit is achieved but should be less than 3%. F2.5C1.0 is not acceptable which shows that in the case of 2.5% bitumen, higher amounts of cement are needed (maybe 2 or 3%) but F4.5C1.0 is acceptable. Looking at the accepted mixes based on this criterion shows that a wide range of different mixes can result from the utilization of M KRC (only the air voids and ITS criteria). Having one catalogue design for all these different types with different behaviors may not be appropriate. The next sections will show how these mixes have different mechanical and performance behaviors under load.

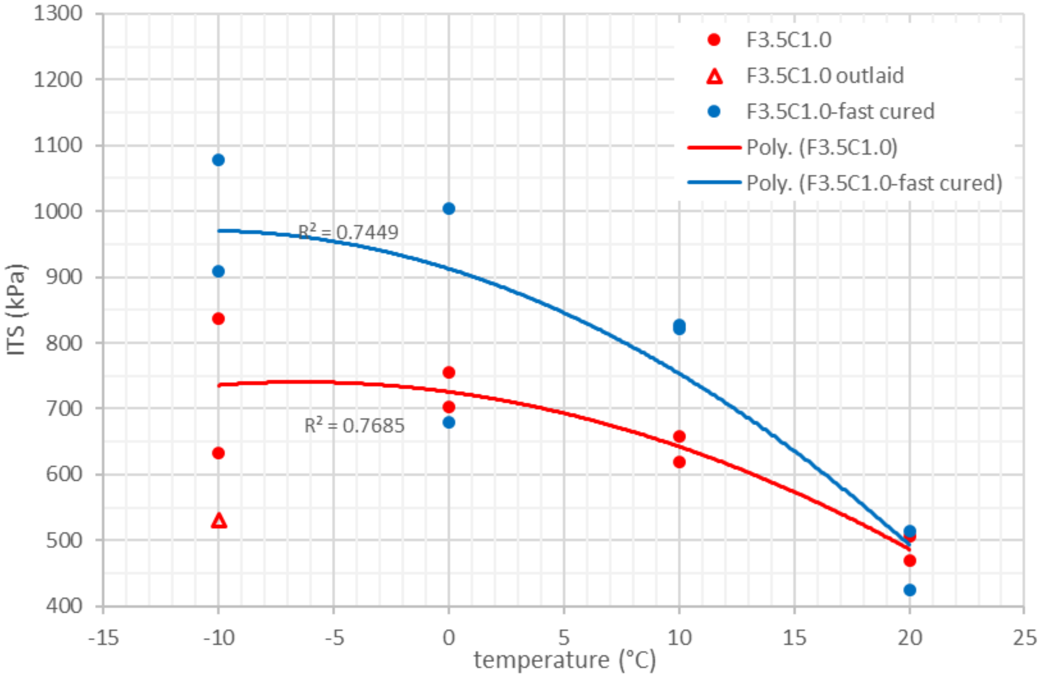


Figure 4-11: Comparing the effect of curing method on ITS test results

Figure 4-11 shows the effect of the curing procedure on the ITS of the F3.5C1.0 mixes at different temperatures. The fast curing procedure (3 days at 40 °C) is compared with the standard curing method. Looking at the graphs, the first point is the high scatter of the ITS results in fast cured samples especially at 0 and -10 °C. It can be related to the variation of the remained moisture after the curing process which leads to different responses at water freezing temperatures.

To figure out the reason for the relatively higher ITS results in the fast cured specimens, the test's data and the fracture surface of the specimens were considered. Figure 4-12, shows the vertical load versus horizontal deformation of the samples during the ITS tests at 10 °C. Looking at the maximum load points shows that the fast cured samples had lower horizontal deformation at the failure point compared to standard cured ones. The higher failure load and lower horizontal deformation reveal that in fast cured specimens, the bonds are stiffer than the standard cured ones. Looking to the test's data of the two fast cured specimens at 0 °C shows that the specimen with lower ITS had a higher amount of horizontal deformation than the other one at failure point (690 µm compared to 204 µm for the one with higher ITS), more in the range of the amounts for the standard cured specimens.

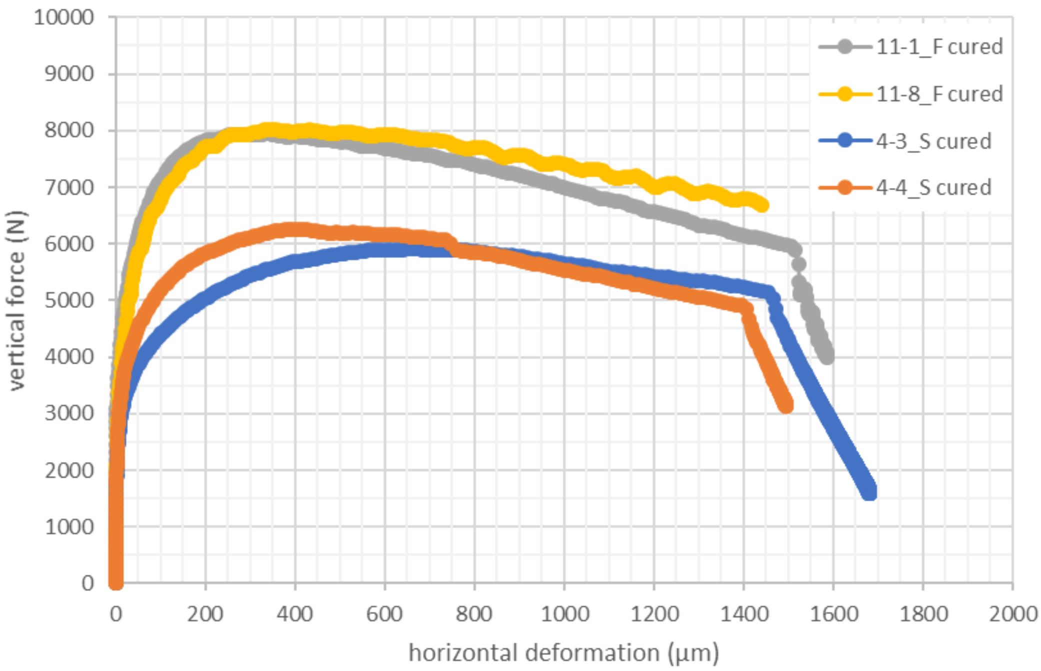


Figure 4-12: Comparison of ITS graphs of standard (S) and fast cured (F) samples (F3.5C1.0) at 10 °C

Figure 4-13 shows how the bitumen mastic phase is distributed through the fracture surface of different cured samples tested at 10 °C. The top two are the standard cured (4-3 and 4-4) and the bottom ones are the fast cured specimens (11-1 and 11-8).

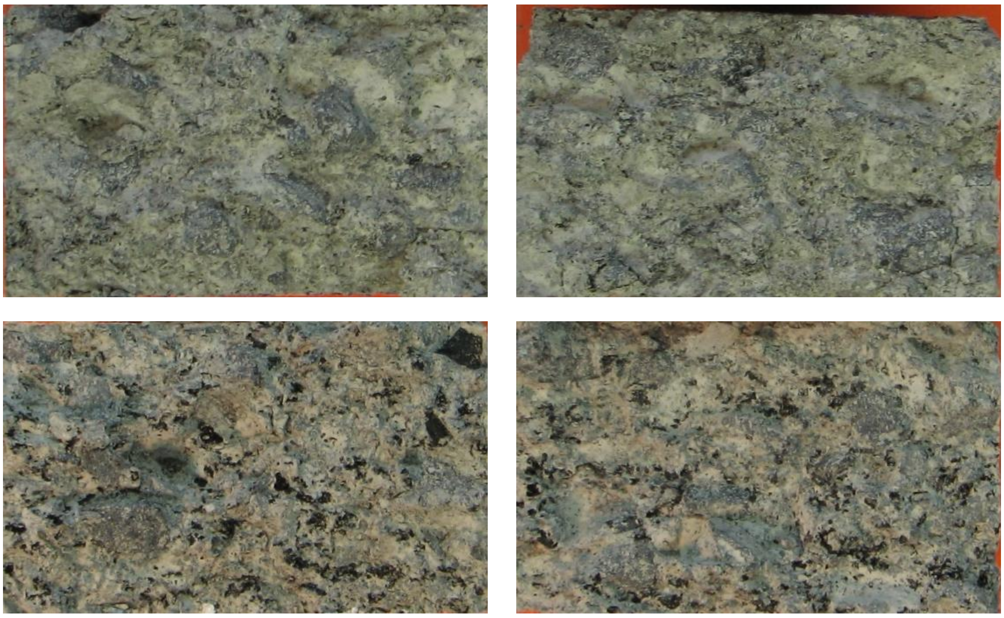


Figure 4-13: Dispersion of bitumen mastic phase on the fracture surface from the ITS tests at 10 °C, top are the standard cured (4-3 and 4-4) and bottom are the fast cured (11-1 and 11-8) specimens (both groups are with F3.5C1.0)

Looking at the pictures clearly shows a higher concentration of bitumen mastic in the fractured surface of the fast cured specimens. This can be one reason for the higher ITS and also more temperature sensitivity of fast-cured specimens. Considering that the failure occurs through the weakest surface, it shows that in these cases, the bonds in fast cured specimens may be stronger than the ones in standard cured ones. Based on the observations, relative humidity in the climate chamber remained high during the 3 days of fast curing. The combination of higher temperature (40 °C comparing to 20 °C) and higher humidity in a longer period (3 days of fast curing compared to the first 2 days of standard curing) can affect the hydration of the cement which makes the cemented filler phase stronger and therefore the fracture line went more through the bitumen mastic phase too. To assess this hypothesis, more test points plus detailed and topic-based concentrated tests are needed which was not the main aim of this research. What can be mentioned is to be sure that the climate chamber (or oven) used for fast curing is capable of extracting the moisture out of the cabin and the specimens reach the constant mass state. This affects the variability of the tests' results.

Looking to these observations on the ITS, deformation characteristics and the fracture plane shows how much this composite material is complicated and how many factors can affect the results of the tests. This shows the importance of considering different aspects during the evaluation and interpretation of the test results and also having enough amount of test points to be able to make a reliable conclusion.

4.4 Poisson's Ratio

The importance of Poisson's ratio, methods to determine it and the amounts that are taken by different researchers were explained in section 2.3.3. In this research, to show and assess the effect of this parameter on the stiffness master curve, two approaches were selected. First the asphaltic model and second, the analytical approach by using the vertical and horizontal measured deformations during the indirect tensile tests. For the asphaltic model, the Witczak equation (see Equation 2-20) was selected to determine the Poisson's ratio at different temperatures.

For the analytical approach, the d_g and c_g constants (see Equation 2-17) were determined by solving the two integrals with 101.6 mm as the samples' diameter, 12.7 mm as the loading strip width and considering the diameter length as the measurement gauge span. The results are d_g = -0.06 and c_g = 3.6. With these constants and having the other two constants, a_g and b_g already, Poisson's ratio was calculated at a range of different horizontal and vertical deformation amounts taken from indirect tensile tests data (up to 45% the maximum load as the limit) at each temperature for all different mix combination. It is important to mention that the vertical deformation was taken from the movement measurements of the loading stamp and horizontal deformation was measured with two LVDTs at the sides of the specimens. The calculated Poisson's ratios were in the range of 0.27 to 0.31 for all the mix combinations and temperatures and mostly around 0.28 (see appendix D). These results show that maybe the assumption of a constant Poisson's ratio for these mixes is acceptable. M KRC [11] guideline also suggests 0.3 which is in agreement with the calculated range. As the sample may have some locally concentrated damages and consequently, vertical deformations in the region of loading strips, it will affect the measurement results of vertical deformation. Therefore, better to measure the vertical and horizontal deformations on a smaller gauge length.

To investigate the effect of Poisson's ratio both models were applied for the calculation of stiffness and master curves of the mix combinations. For the constant amount model, 0.28

was selected as Poisson's ratio. Based on the Witczak equation the amount of Poisson's ratio will be 0.17 for -10 °C, 0.2 for 0 °C, 0.22 for 5 °C, 0.24 for 10 °C and 0.30 for 20 °C rounded to 2 digits after zero.

4.5 Horizontal strain at failure (strain at break)

As mentioned before, it is possible to trace the effect of binding agents on the mechanical characteristics of the mixture by studying the force-deformation graphs and calculating different indexes and parameters from them. As this topic was not in the main scope of this research and the number of the specimens and test points are not enough for a reliable conclusion, only the horizontal strain at failure (the point of maximum load in the ITS test) was calculated for one temperature (20 °C) and with one Poisson's ratio (equal to 0.28) for all mix combinations to see if this parameter is appropriate to trace the possible change in mechanical characteristics of the FCSMs because of the amount of binding agents or not. Figure 4-14 shows the changes in the amount of strain at failure for different cement contents. It can be seen that the scatter of the results is high in the mixes with 1% cement but considering the whole data points, there is a shift to the left by the increase of the cement which is with an increase in ITS and decrease in the strain at failure parameter. The high variability in the results was reported by Twagira too [30].

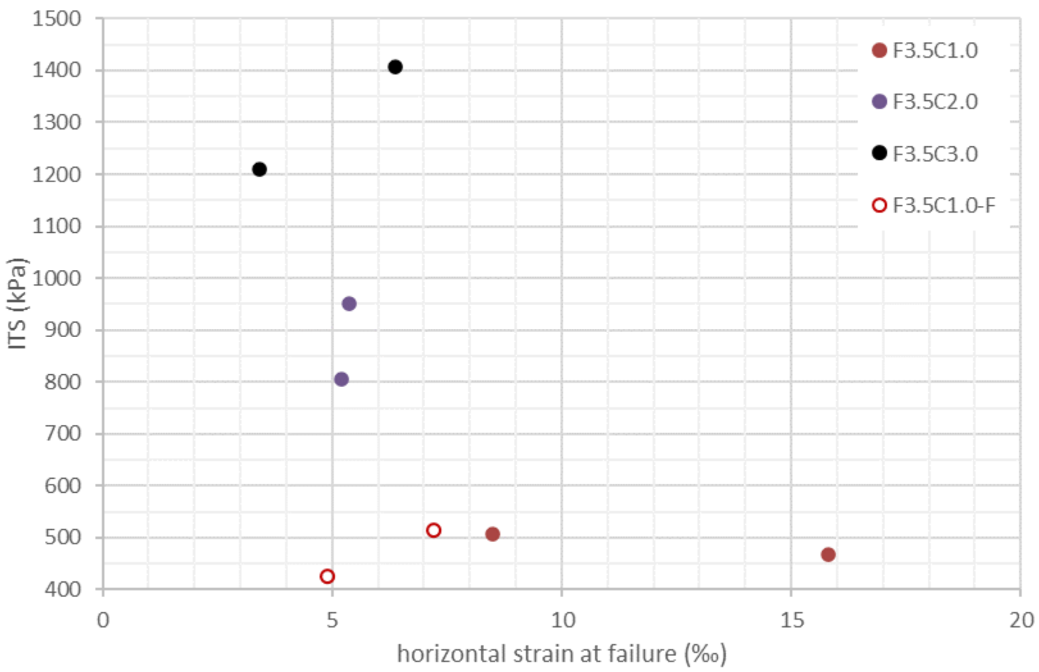


Figure 4-14: The effect of cement content on horizontal strain at failure from ITS tests at 20 °C (F, stands for the fast cured method)

Another point is that the fast cured samples with 1% cement content are between the results of standard cured samples with 1 and 2% cement which rudimentary confirms the hypothesis of better hydration of cement with this curing method.

4.6 Stiffness and the master curves of the mixes

The results and findings from different cyclic stiffness tests on different mix combinations will be presented in this section. As mentioned already the main goal was to assess the effect of binding agents on the stiffness characteristics of this material. Besides this main goal, some secondary topics like the effect of strain (or stress) level on the tests' results, resilient behavior of the stiffness and the effect of aggregate orientation on the results, which are related to the stiffness theme will also be noticed and will be discussed too.

As the aim of cyclic stiffness sweep tests were to investigate the effect of binding agents on the response of FCSMs at different temperatures and frequencies, it is logical to perform the tests in an appropriate range of horizontal strain which both of the main bond types (of the two different binders) can be reasonably assumed to be intact. Besides that, the specimens were decided to be used for cyclic fatigue tests too, so it was also important to be sure that they are not damaged. The third point to be considered was that if a sample were damaged during the first frequency level test, then for each frequency and temperature separate samples are needed. Considering the amount of different temperatures and frequencies (16 combinations) and the repeating for each, the number of samples for one round of stiffness sweep test becomes unpractical. Because of these three reasons the tests were performed at strain levels lower than the common amounts for HMA specimens. This approach is common as mentioned in the literature review (see section 2.3.3) and also was seen during the indirect tensile tests. The selected specimens were divided into three groups and each group was used for one temperature and all frequencies. As there were 4 temperatures, therefore one group was used for two temperatures. At each temperature after the end of the frequency sweep round, a control test was performed at the first frequency to be sure that the specimen is not unacceptably damaged. Each stiffness test was started with a relatively low stress level. Then the amounts of horizontal deformation, horizontal strain and the ratio of applied stress to the ITS amount, were monitored. The decision to stop or continue the test with a higher stress level was made based on these parameters. 2 μm horizontal deformation and around 30% of ITS were used as guiding limits.

AL SP Asphalt 09 guideline [107] was used to calculate the stiffness and later to construct the stiffness master curves. As mentioned before, since 2019, there is a new guideline available for HMA stiffness tests in Germany (TP Asphalt-StB Teil 26) [187]. AL SP Asphalt 09 guideline uses Arrhenius equation (with 25,000 as the constant) to determine the shift factors (α_T) and later the Sigmoidal function as regression curve. The same method was applied by Jenkins and Twagira (they used 10,920 instead of 25,000 as the constant) [30]. Godenzoni et al. [51] applied the Heut-Sayegh model and mentioned the time-temperature superposition is valid for the foamed bitumen mixes too. Kowalska and Maciejewski [57] applied the Williams-Landel-Ferry equation to determine the shift factors and the Sigmoidal function for the regression model. The new German guideline uses the Sigmoidal function but with upper and lower limits for stiffness which are determined based on the glass module by using the phase angle data of the tested specimens in some test temperatures. This approach assures that at high temperatures the stiffness won't become negative.

After calculation of the stiffness at each temperature and frequency, the reasonable ones were selected for the construction of the master curves. To select the points from the results of 3 samples, besides considering the control test (to see if the specimen was intact), the amount of CV (coefficient of variation between the 3 results) and having a logical shape and order of the iso-thermic graphs were considered altogether. 20 °C and 10 Hz were the base to construct the master curves. Figure 4-15 shows the master curve of F3.5C1.0 mix with the asphaltic model Poisson's ratio, as an example.

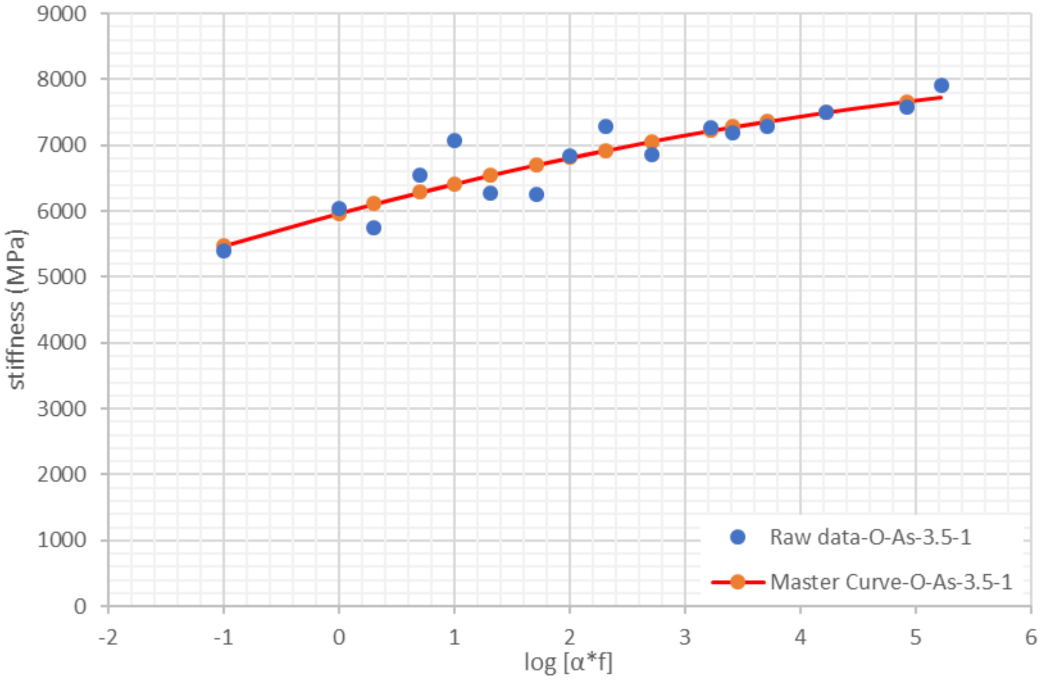


Figure 4-15: Master curve of the F3.5C1.0 mix (20 °C and 10 Hz as the reference, As stands for asphaltic Poisson's ratio model and O for original data points)

To determine the stiffness at any temperature and frequency first the shift factor (α_T) at that temperature will be calculated (Equation 4-1). By having the frequency, the log (α_T *f) is determined and with the help of the master curve equation (as the x in Equation 4-2), the stiffness is calculated at the desired temperature and frequency.

$$\alpha_T = e^{25000\left(\frac{1}{T}-\frac{1}{T_0}\right)}$$ **Equation 4-1**

In Equation 4-1, T_0 is the selected base temperature for the master curve in Kelvin (here is 20 °C, equal to 293°) and the T is the desired temperature in Kelvin. The Sigmoidal function of the stiffness master curve and its parameters are:

$$E = y_0 + \frac{w}{1 + e^{-\left(\frac{x-x_0}{z}\right)}}$$ **Equation 4-2**

E, stiffness at desired temperature and frequency (MPa)

y_0, intersection of the function with the ordinate axis (MPa)

x_0, intersection of the function with the abscissa axis (Hz)

w, material's parameter (MPa)

z, material’s parameter (-)

Figure 4-16 shows the stiffness of the F3.5C1.0 mix, calculated for different temperatures and 10 Hz frequency based on the master curve equation, as an example. Looking at the master curve shows that the stiffness of the mix has temperature and loading frequency dependency which is logical because of the existence of bitumen.

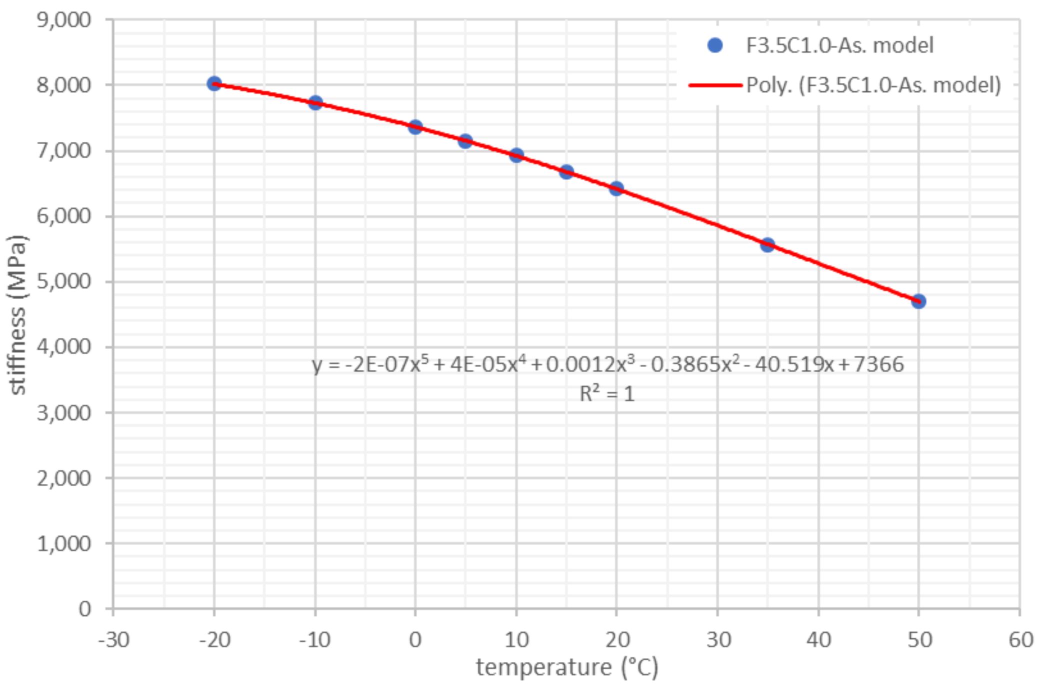


Figure 4-16: Stiffness of F3.5C1.0 (different temperatures, 10 Hz) calculated from master curve (As stands for asphaltic Poisson’s ratio model)

Looking to the reports of Godenzoni et al. [31], Kowalska and Maciejewski [57] and Schwartz et al. [192], the general shape of the stiffness master curve agrees with the ones depicted there. Nataatmadja [115, 69] reported modulus values of 12,000 to 14,000 MPa for Marshall compacted specimens with 2.5% foam bitumen and 2% lime. In New Zealand depending on the type of bitumen, Saleh [68] reported amounts ranging from 4,300 to 8,600 MPa (at 20 °C) for granular material stabilized with 3.5% foam bitumen and 1% cement. Gonzalez [8], found the same trend during his tests with the indirect tensile method and reported resilient modulus values of 5,500 to 7,500 MPa for mixes with 2% foam bitumen and 1% cement and 6,500 to 8,000 MPa for mixes with 3% foam bitumen and 1% cement (at 20 °C) with maximum values with 2.5% foam bitumen (around 7,000 MPa). In this research, the range of stiffness amounts for F3.5C1.0 specimens is between 6,000 to 8,000 MPa (at 20 °C, 10 Hz). Based on the master curve regression, the amounts are 6,400 and 6,700 MPa (depending on Poisson’s ratio model). For F2.5C1.0, the amounts range from 4,000 to 5,000 MPa with 5,600 and 5,900 MPa from the stiffness master curves (depending on Poisson’s ratio model). The results show a good agreement with the literature and can be proof of them. The range of stiffnesses for F3.5C2.0 mixes are from 11,500 to 14,000 MPa (at 20 °C, 10 Hz) and compared to the amounts reported by Nataatmadja, shows how the higher amounts of hydraulic binders can mask the effect of bitumen.

Figure 4-17, shows graphically the stiffness master curves of different mix combinations. individual stiffness results and the graphs are presented in appendix E.

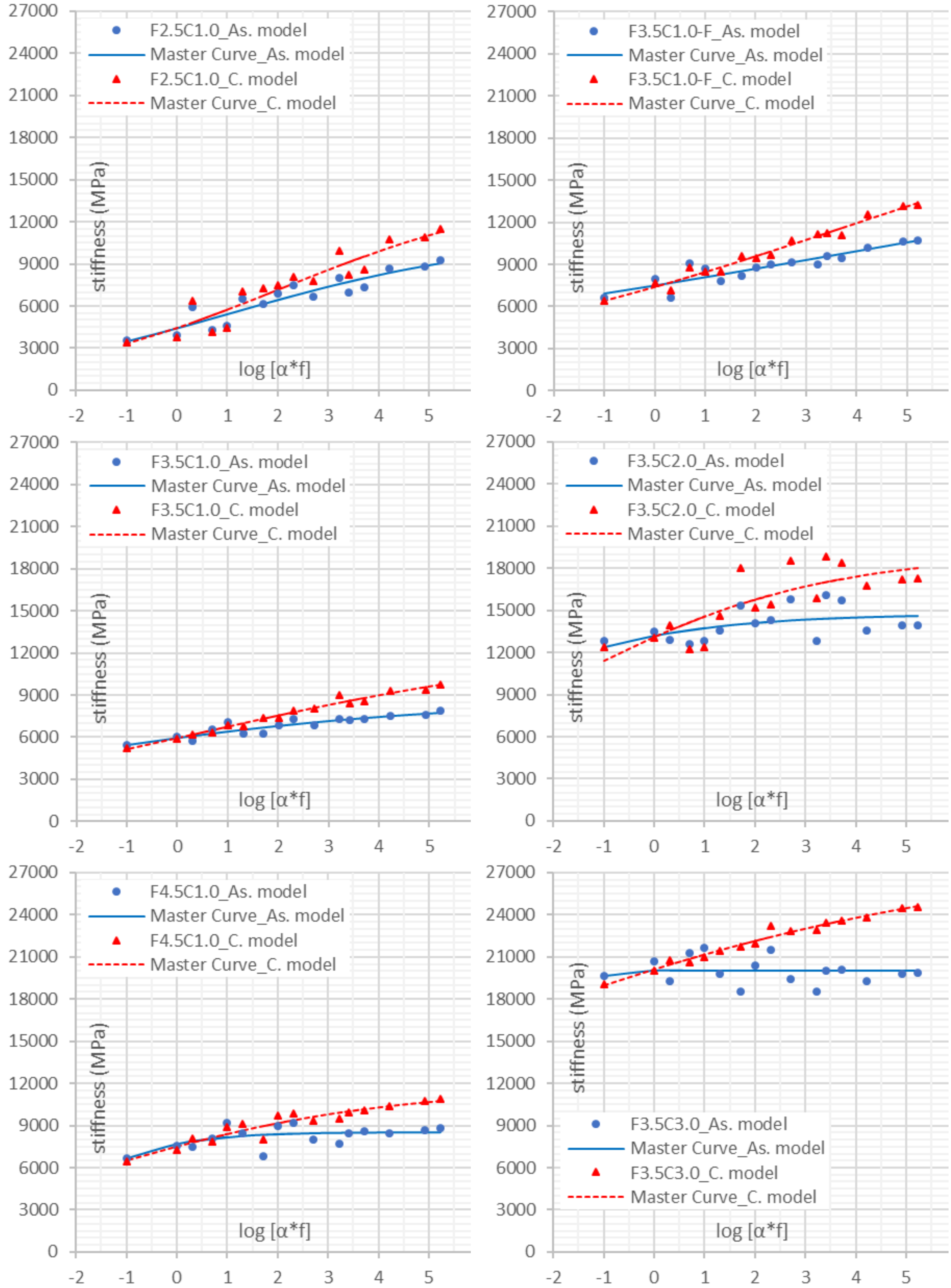


Figure 4-17: Stiffness master curves of different mix combinations with 2 Poisson's ratio model (As. stands for asphaltic model and C. stands for the constant model equal to 0.28; the F. letter after the mix combination shows the fast cured method)

The aim was to show how Poisson's ratio parameter can affect the resulted mater curve and conclusions about mixes' characteristics. The constant Poisson's ratio model affects mostly the higher frequency (or lower temperature) part and increases the amount of stiffness compared to the asphaltic model which is temperature-dependent. The Poisson's ratio sharply decreases in lower temperatures (-10 °C). It is important to mention that the real master curve is between these two models but to determine which model is more acceptable, special tests with the aim of only measuring the Poisson's ratio are needed to be performed. The other point from the picture is: in the range of conventional mixes (bitumen 2.5 to 3.5% and cement around 1%) the effect of Poisson's ratio model is not too big compared to the higher amounts of cement and bitumen. Table 4-6 shows the master curve equation parameters of the mix combinations with two different Poisson's ratio models mentioned before.

Table 4-6: Master curve equation parameters of different mixtures with 2 Poisson's ratio models (F. letter after the mix combination shows the fast cured method)

Mix Type	Asphaltic model Poisson's ratio				Constant Poisson's ratio (= 0.28)			
	y_0 (MPa)	x_0 (Hz)	w (MPa)	z (-)	y_0 (MPa)	x_0 (Hz)	w (MPa)	z (-)
F2.5C1.0	0.001	1.0214	10931.078	2.700	0.001	2.0512	14469.006	2.507
F3.5C1.0	0.001	-3.1318	8638.118	3.897	0.001	0.4316	12532.171	3.858
F3.5C1.0-F	0.001	4.6059	20609.324	8.239	0.001	3.8863	23648.621	4.964
F4.5C1.0	0.001	-2.3353	8517.781	1.060	0.001	-1.6767	11683.775	2.822
F3.5C2.0	0.001	-4.4587	14738.101	2.069	0.001	-1.9915	19118.199	2.571
F3.5C3.0	0.001	-1.3229	20011.378	0.079	0.001	-4.7242	28419.863	5.339

The effect of cement on the stiffness can be seen by looking at the changes in parameter w (Table 4-6), which increases with cement increase but decreases with bitumen increase. The difference between 3.5 and 4.5% is not big; because increasing the bitumen amount, changes the size of the bituminous mastic droplets (to bigger sizes) and leads to a poorer distribution of them through the mix. This poorer distribution affects the mixes' characteristics and therefore the overall result. Fu & Harvey [117], applied a simplified equation (Equation 4-3) to transfer the stiffness of foamed bitumen mixes from a measured temperature to a target temperature. It is used to correct the back-calculated stiffnesses from FWD measurements to a standard temperature (20 °C).

$$E(T_0) = 10^{\propto(T-T_0)}E(T)$$ **Equation 4-3**

$E(T_0)$, stiffness at desired target temperature (MPa)

$E(T)$, stiffness at measurement temperature (MPa)

T_0, target temperature (°C)

T, stiffness measurement temperature (°C)

α, temperature coefficient (1/°C)

The temperature coefficient is a good parameter to assess the effect of cement content on the temperature dependency of the foamed bitumen mixes. To do that, the stiffness master curves of mixes with 3.5% bitumen but different contents of cement were considered. The stiffness of each mix combination was determined at different temperatures and 10 Hz frequency. By considering the stiffness at 20 °C as the target, the Excel solver was applied to determine the temperature coefficient (α) for each mix combination. Figure 4-18 shows the amount of α versus the cement content of the mix. It is also possible to see the effects of Poisson's ratio model and the curing method on the temperature coefficient too. For the zero cement content, the reported amount from Fu & Harvey (as 0.0131) [117] was added to the graph.

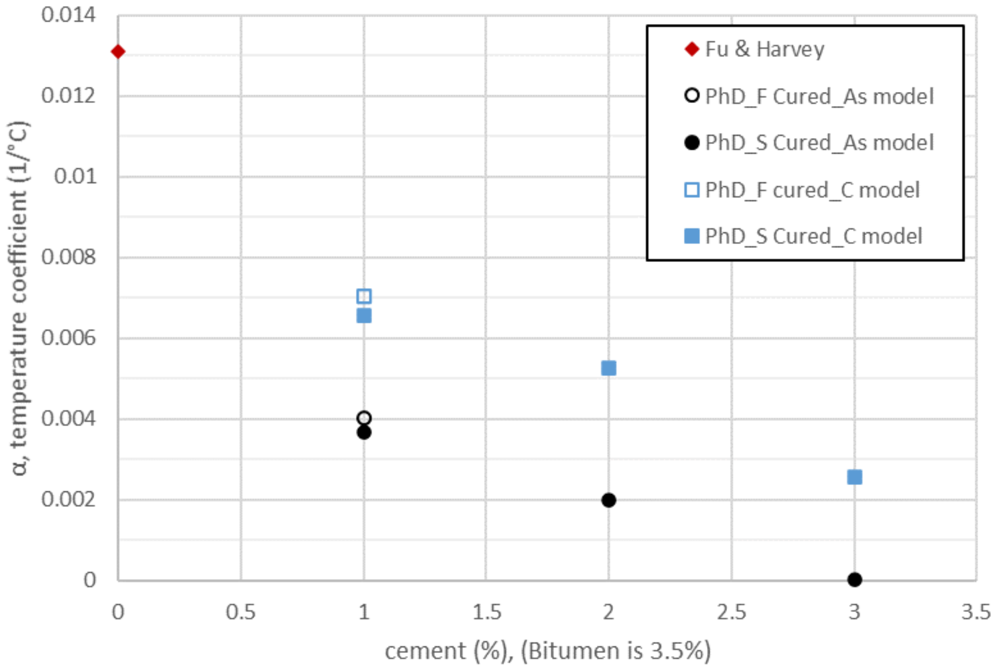


Figure 4-18: Temperature coefficient parameter change versus the mixes cement content (As. stands for asphaltic C. stands for the constant Poisson's ratio models; the F. shows the fast and S, the standard curing methods)

Looking to the trend of the points in Figure 4-18 shows that:

1- Independent from the applied Poisson's ratio model, the temperature coefficient parameter (α) decreases with the increase of the cement content. This proves the decrease of the temperature dependency of the foamed bitumen mixes by increasing the cement content. It is important to notice that depending on Poisson's ratio model, the amount of temperature dependency will differ. As can be seen in the figure, the constant Poisson's ratio model shows temperature dependency even in the mix with 3% cement whereas the asphaltic Poisson's ratio model shows no temperature dependency in the case of 3% cement. This again reveals the importance of Poisson's ratio on the interpretation of the cyclic indirect tensile test results.
2- Comparing the temperature coefficient of the standard and fast cured specimens together, independent of Poisson's ratio model, the amounts are almost the same. This shows that the curing method doesn't affect the temperature dependency of the foamed bitumen mixes with 1% cement.

Figure 4-19, shows the stiffness-temperature graphs of the mixes determined by calculating the shift factor and using the master curve.

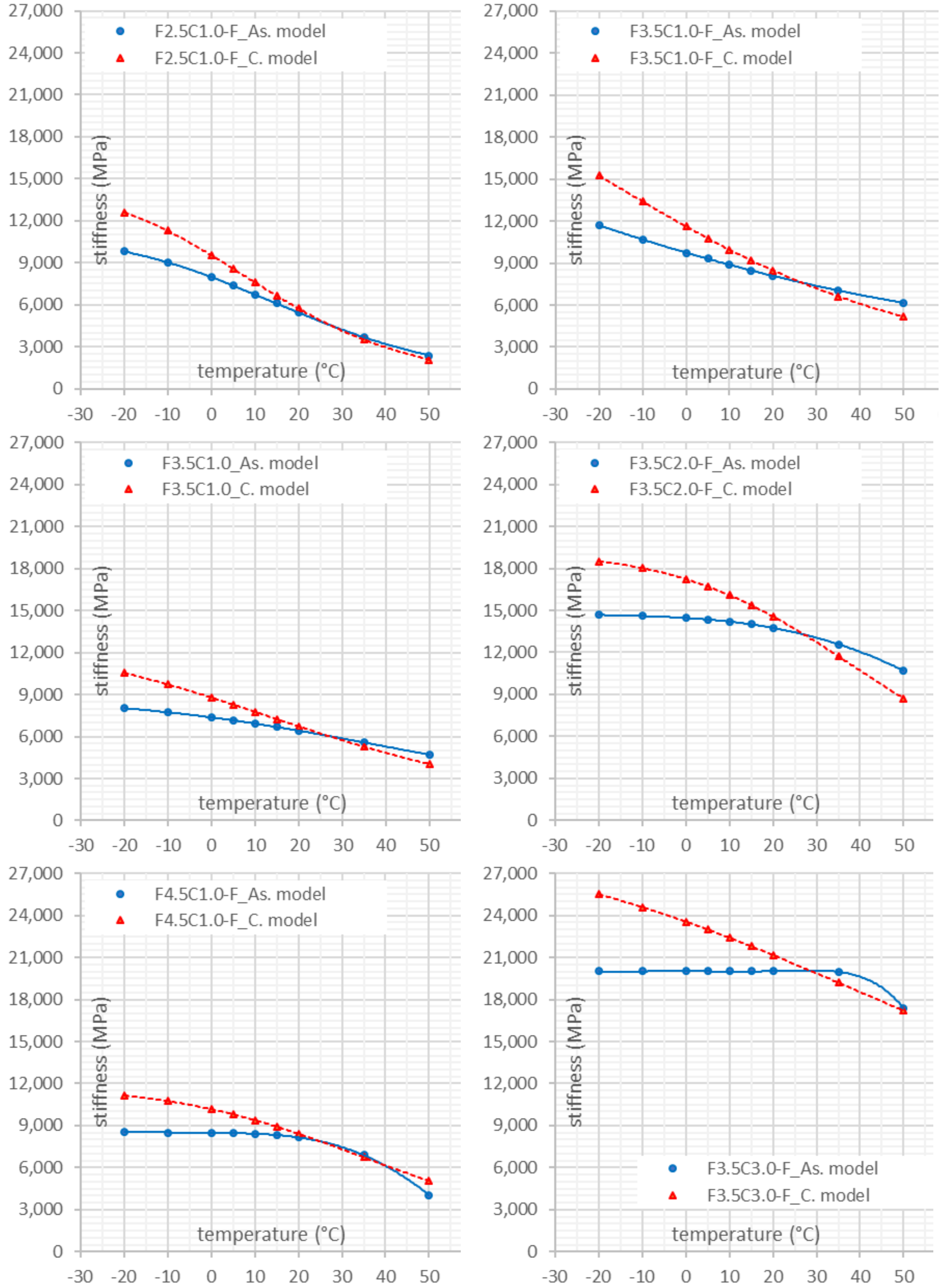


Figure 4-19: Stiffness of different mix combinations at different temperatures (10 Hz) with 2 Poisson' ratio models (As. stands for asphaltic model and C. stands for the constant model equal to 0.28; the F. letter after the mix combination shows the fast cured method)

Again, to show the effect of Poisson's ratio assumption model, all curves are with both models. The amount of stiffness determined by the Sigmoidal model for the F3.5C3.0 at 50 °C with asphaltic model Poisson's ratio, became zero and the amount suggested based on AL SP Asphalt 09 guideline equation wasn't logical, therefore for this case a polynomial curve with order 5 was fitted to the data and used for determining the stiffness at 50 °C. All the other mix combinations had logical and acceptable results (nearly double the control one) at 50 °C with AL SP Asphalt 09 procedure. The only way to see which type of equation is better to be used for this material is to perform a set of stiffness tests at higher temperatures and frequencies. Table 4-7 shows the graphs data points.

Table 4-7: Stiffness of different mix combinations at different temperatures (and 10 Hz) with 2 Poisson's ratio models (As. stands for asphaltic model and C. stands for the constant model equal to 0.28; the F. letter after the mix combination shows the fast cured method)

Poisson's ratio & the Mixes		Stiffness amount (MPa) at different temperatures (°C)								
		-20	-10	0	5	10	15	20	35	50
As. model	F2.5C1.0	9,800	9,026	7,983	7,381	6,744	6,092	5,444	3,686	2,376
	F3.5C1.0	8,019	7,732	7,366	7,154	6,924	6,677	6,416	5,573	4,702
	F3.5C1.0-F	11,701	10,690	9,746	9,302	8,877	8,471	8,085	7,039	6,150
	F4.5C1.0	8,516	8,511	8,489	8,462	8,412	8,322	8,166	6,893	4,052
	F3.5C2.0	14,676	14,602	14,459	14,347	14,199	14,004	13,755	12,587	10,706
	F3.5C3.0	20,011	20,011	20,011	20,011	20,011	20,011	20,011	19,983	17,365
C. model (0.28)	F2.5C1.0	12,611	11,284	9,547	8,584	7,605	6,646	5,740	3,510	2,069
	F3.5C1.0	10,537	9,723	8,780	8,276	7,760	7,242	6,727	5,272	4,037
	F3.5C1.0-F	15,256	13,406	11,617	10,768	9,960	9,196	8,480	6,620	5,168
	F4.5C1.0	11,141	10,751	10,176	9,811	9,394	8,929	8,422	6,739	5,058
	F3.5C2.0	18,524	18,029	17,243	16,717	16,096	15,378	14,568	11,730	8,730
	F3.5C3.0	25,504	24,602	23,567	23,004	22,416	21,804	21,174	19,206	17,207

Looking at the curves at higher cement amounts (2 and 3% cement) it seems that apparently, the asphaltic Poisson's ratio model led to more logical results as the stiffness is expected to be less temperature-dependent at these cement amounts. But it is important to consider that the mixes still have 3.5% bitumen and the constant Poisson's ratio graph shows that this amount of bitumen resulted in temperature dependency even with 3% cement. As mentioned before, the real curve may lay between these two models and still shows some amount of temperature dependency of the stiffness for these mixes.

To sum up, considering the logical results of increasing the bitumen and cement and the changes in material's microstructure, the accurate Poisson's ratio amount is a temperature-dependent model (because of the existence of bitumen) but the temperature dependency is less than the asphaltic model (i.e., Witczak) because of the inclusion of cement as the second binder. The equation may be dependent on the amount of cement too. The aim of this research was not to solve the Poisson's ratio issue but to open it as a point that needs to be taken into account especially when higher than normal ranges of cement or bitumen are applied.

Figure 4-20 shows all master curves together with the asphaltic Poisson's ratio (upper graphs) and the constant Poisson's ratio (lower graphs).

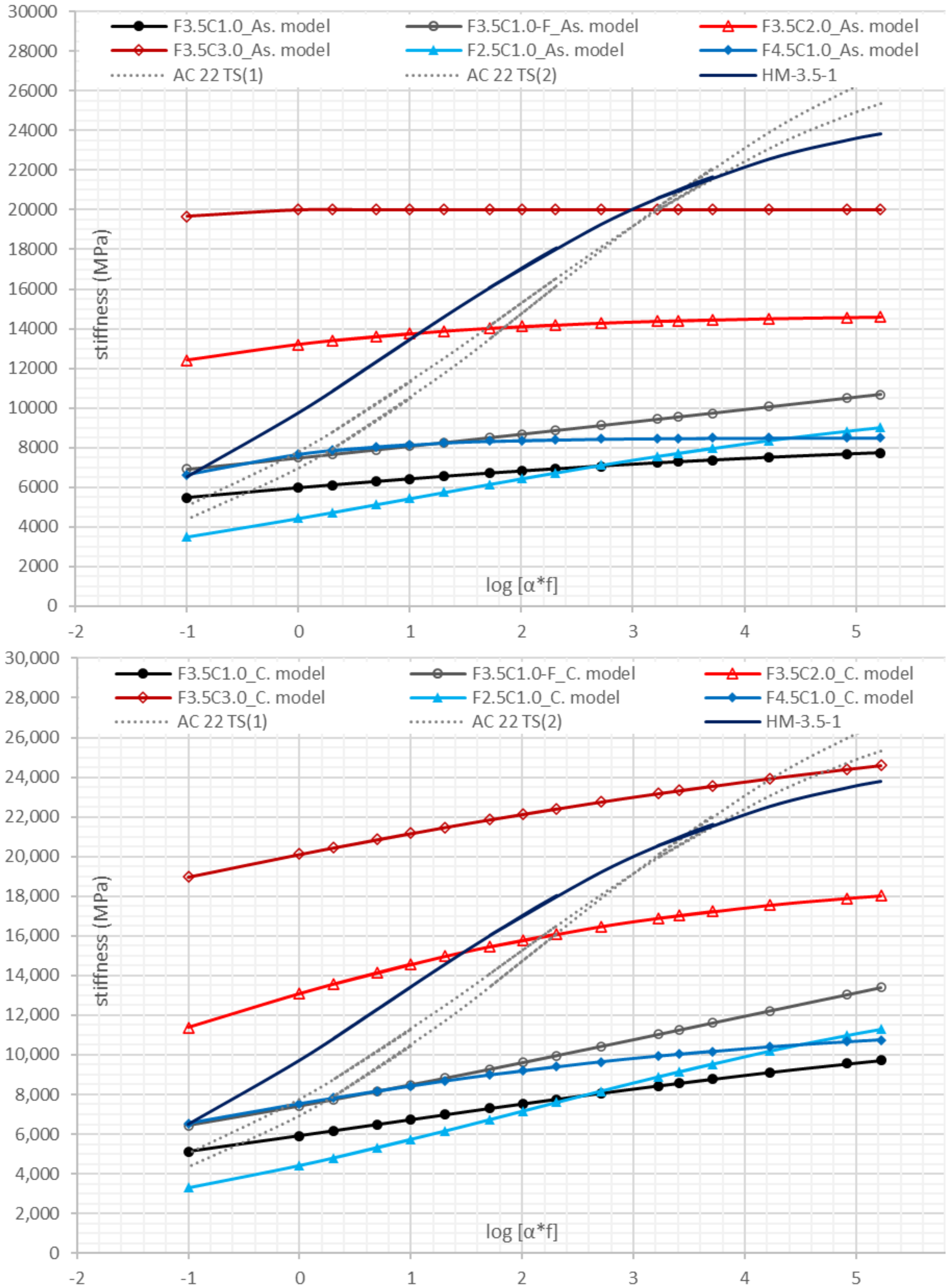


Figure 4-20: The effect of binding agents' content and the production method on the resulted stiffness master curves of the mixes (base temperature is 20 °C; As. stands for asphaltic model and C. stands for the constant model equal to 0.28; the F. letter after the mix combination shows the fast cured method)

To have a reference, a series of specimens with the same material, mix design, samples size and compaction method as F3.5C1.0 were produced (batch 23) with the only difference that instead of foaming bitumen and cold mixing process, the hot mix asphalt procedure was used (the master curve is named with HM-3.5-1). The parameters of the sigmoidal function master curve equation for this mixture are: y_0 = 0.001 MPa, x_0 = 0.8310 Hz, w = 25,628.594 MPa and z = 1.7037. Two master curves of normal hot mix asphalt base were added to the picture for comparison too. The first impression from the picture is that independent of Poisson's ratio model, FCSMs are considerably less temperature-sensitive than the hot mixes which has been mentioned by different researchers in the literature too. This is because of the nature of bitumen coating and also the existence of cement.

Normally in conventional HMA, the stiffness is time and temperature-dependent. looking at the stiffness master curve made based on indirect tensile testing, stiffness of bitumen at low temperatures limits the stiffness of the mix while at higher temperatures, the aggregate's pack skeleton achieved in the compaction process dominates the lower stiffness limit. It is important to note that, the amount of bitumen can affect the compaction and the aggregate packing and then indirectly affect this lower limit. In the case of FCSM, cement as the second binder affects both higher and lower stiffness limits. It can also affect the rate of temperature dependency too. The other difference comes from the nature of bitumen coating and distribution which is completely different in foamed bitumen cold mixes compared to hot bituminous mixes. These non-continuous bonds from the bituminous mastic phase affect the temperature dependency of the resulted mix which is less influenced by bitumen amount.

Comparing the master curves of F3.5C1.0 with the same hot made one (HM-3.5-1) at 50 °C, the stiffness of F3.5C1.0 is between 4,000 to 4,700 MPa (see Table 4-7, depending on the Poisson's ratio) and is 3,280 MPa for HM-3.5-1 (determined by the master curve equation for 50 °C temperature and 10 Hz frequency). This shows the combined effect of cement which is argued to have a positive effect on material's resistance to rutting at higher temperatures. The considerable difference on the low-temperature side shows the above-mentioned point about the influence of bituminous bonds' non-continuity on time and temperature dependency. The size of mastic phase droplets and their distribution throughout the mix doesn't have a unidirectional dependence on the amount of foam bitumen in the mix. Higher amounts lead to bigger mastic phase droplets which increases the heterogeneity of the mix because of poorer distribution. In the case of cement, increasing its content will bond the filler particles and stiffens that phase which leads to an overall stiffness increase. It also affects the bituminous mastic phase stiffness and its temperature sensitivity too.

Comparing the effect of cement content with the effect of bitumen amount, clearly shows how cement increases the stiffness and shifts the mix to the hydraulically dominant side. Based on the results, more than 1% cement leads to more hydraulic dominant mixes which agree with the general recommendations in the literature to limit the amount of cement or use hydrated lime in cases that more flexible mixes are desired. It seems that 8,000 MPa (at 20 °C,10 Hz), can be mentioned as a limit for changing to hydraulically dominant mixes. The same as this trend of cement effect was seen in the ITS graphs too. It shows that maybe ITS can be a good index for fast ranking of different mixes. These observations confirm the reported results from the CoRePaSol project too [193].

It can also be seen that the fast cured samples show higher stiffness compared to the standard cured one, this was observed already in ITS results too. As explained before, better hydration of the cement can be the main reason for this. But the graph is still far from the F3.5C2.0 (the one with 2% cement).

The general shape of the stiffness master curves (Figure 4-20) matches well with the results of the NCHRP research project on different stabilized / recycled projects with foamed bitumen and cement, reported by Schwartz et al. (figures 17 and 18 on pages 27 and 28 of the report) [192]. They have also reported that the inclusion of cement leads to a decrease of stiffness at higher frequencies compared to the mixes without cement (figure 20, page 29 of the report). They reported higher CVs (coefficient of variations) between the tests' results at higher test temperatures and lower test frequencies and mentioned that the range of 9 to 24% which is suggested by AASHTO TP 79, needs revision when stabilized / recycled mixes are evaluated.

4.6.1 Assessing the effect of the master curve construction method

To assess the effect of the master curve construction methods on the resulted stiffness master curves, two different methods of AL Sp Asphalt [107] and the TP Asphalt-StB Teil 26 [187] were compared together for the F3.5C1.0 mix. As mentioned before, in AL Sp Asphalt, shift factors (α_T) are calculated based on the Arrhenius type equation and with 25,000 as the constant (see Equation 4-1) but TP Asphalt, uses another method for calculation of the constant instead of using 25,000. Jenkins and Twagira used the amount of 10,920 instead of 25,000 [30]. The TP Asphalt also calculates the $E_{max.}$ based on the phase angles data. To assess the effect of these two parameters on the resulted stiffness master curves, different options were considered:

1- The AL Sp Asphalt with the shift factor constant of 25,000 as the base method;
2- The TP Asphalt method with the $E_{max.}$ to be calculated based on the phase angles (equal to 28,343 MPa);
3- The TP Asphalt method with the $E_{max.}$ selected based on the range from the first method (equal to 12,000 MPa);
4- The AL Sp Asphalt with the constant determined from the third option (equal to 13,688);
5- The AL Sp Asphalt with the constant determined from the second option method (equal to 12,742);
6- The AL Sp Asphalt with the constant from Jenkins and Twagira (equal to 10,920).

Considering the above-mentioned options, all used the Arrhenius equation type for the shift factors but with different constant amounts as the variable and also all used the Sigmoidal function for the stiffness master curve but with different $E_{max.}$ amounts. Figure 4-21 shows the resulting master curves. For better comparison, the master curve of the hot-made samples (HM-3.5-1) has been added to the graphs too. Comparing the different methods with the base method shows some differences but the amount is not big and is mostly on the higher frequency side. Comparing them to the HM-3.5-1 shows the lower temperature sensitivity as mentioned before. This comparison shows that these variables may not lead to a considerable difference between the constructed master curves.

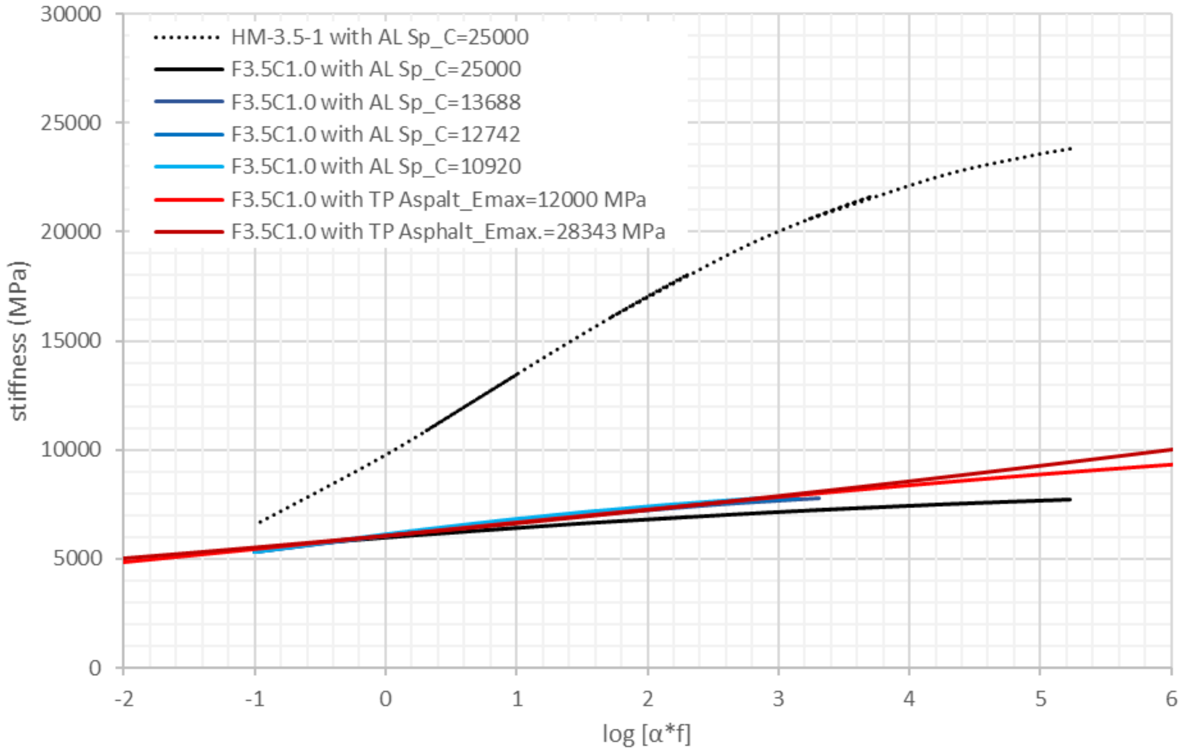


Figure 4-21: Comparing the stiffness master curves constructed with different methods for F3.5C1.0 mix together and with the HM-3.5-1 (all the stiffnesses are calculated with the asphaltic model Poisson's ratio)

4.7 Phase angle analysis

The phase angle is the difference between the vertical load and the horizontal deformation during the cyclic stiffness tests. The amount changes with the test temperature and loading frequency, reflecting the viscous behavior of the binder in bituminous mixtures. Normally higher testing temperature or lower loading frequency leads to an increase of the phase angle but it decreases at lower temperatures and higher loading frequencies. The range of a mixture's phase angle (as the difference between the maximum and minimum amounts) shows the level of the material’s viscous behavior. It can be used to study the effect of bitumen and cement contents on the viscous characteristics of the FCSM. Figure 4-22, is made based on this idea from the calculated phase angles during the stiffness tests. The amounts of the phase angles at different temperatures and loading frequencies during the stiffness tests (temperature ranges from 20 to -10 °C and frequency from 10 to 0.1 Hz) were considered and the maximum and minimum values were taken to show the range of the parameter for each mix type. To be able to compare the hot and cold production effect, this range is shown for the hot-made samples with the same mix design as F3.5C1.0, too. The first three bars in Figure 4-22 from the top, show the effect of bitumen; as can be seen, by increasing the bitumen content, the range of phase angles increases which shows more viscous behavior. The effect is not big from 2.5 to 3.5% bitumen but is more in the case of 4.5% bitumen. The second three bars show the effect of cement. The range is almost the same, but the minimum and maximum amounts decreased in each step of increasing the cement. It shows that the mixes become considerably less viscous but still have temperature (and loading rate) dependency because of the existence of bitumen. This was seen in master curves too. The last two bars show the difference between hot and cold production. It confirms that the foamed bitumen cold mixes are less viscous than their hot bituminous counterparts even on the same mix design and compaction method. the reason is the difference in bitumen dispersion between these two types which results in different viscous behaviors even with the same bitumen content. The same observations were reported by Kuchiishi et al. [194] too.

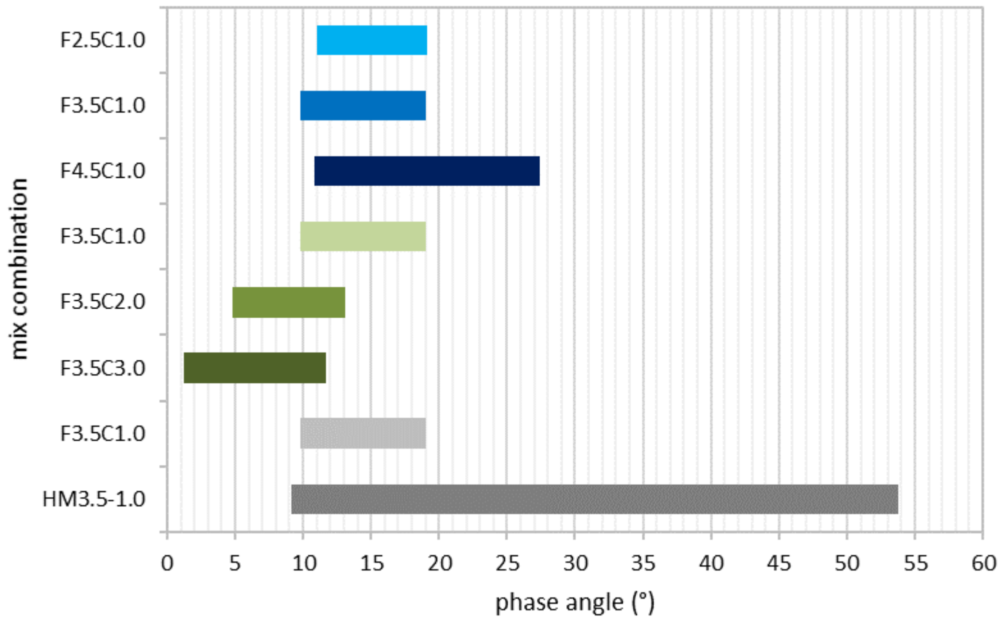


Figure 4-22: Phase angle range of different FCSM mixes compared together and with the hot made asphalt (from the indirect cyclic stiffness tests at different loading frequencies and temperatures)

4.8 Effect of the test's strain level on the amount of stiffness

As mentioned already in the literature review, there isn't any fixed or standard strain range for the indirect tensile stiffness tests on FCSM. Because of the already mentioned reasons, for this research, a relatively low range (compared to the normal range for HMA) was adopted. To see the effect of the test's stress level (or the horizontal strain level which is the result of that) on the resulted stiffness from the indirect tensile cyclic test and later to investigate the effect of possible damage on the sample's mechanical characteristics, a series of tests were planned and performed on different samples in three different steps as follow:

Step one: Multi-step stiffness tests

Step two: ITS tests on intact samples and the samples from step one tests

Step three: Multi-round stiffness tests

4.8.1 Multi-step stiffness tests

A series of samples were tested at different strain levels. Started with inserting low stress to have a low amount of horizontal strain around 0.001‰ and then increasing that step by step and measuring the stiffness up to high amounts of horizontal strain (0.11‰ to 0.12‰). The procedure is named multi-step stiffness tests. The aim was to see the effect of strain level on the resulting stiffness. Two batches with two different mix combinations of F3.5C1.0 and F2.5C1.0 were produced and cured with the standard method. The samples of each batch were then divided into two groups and one group from each mix combination was used for multi-step stiffness tests (named as group B with 5 and 6 samples for each of the mix combinations respectively). The second group (group A) was stored for the next test which is explained later. Stiffness tests were performed at 5 °C and 10 Hz. Poisson's ratio was calculated from the asphaltic model (0.216 at 5 °C). A regression was done on the results of each sample to determine the amount of stiffness at other strain levels. As different samples have different strains at the same stress, the equation will be used to determine the amount of stiffness at the same strain levels. Then the data of all samples were used to determine the stiffness amount at each strain level with 95% and 99% levels of confidence which was later used to evaluate the effect of strain level on the resulting stiffness. To control the level of accuracy of the models, the predicted amounts from the models were compared to the results from the tests on the same mix combinations and test conditions but with other samples.

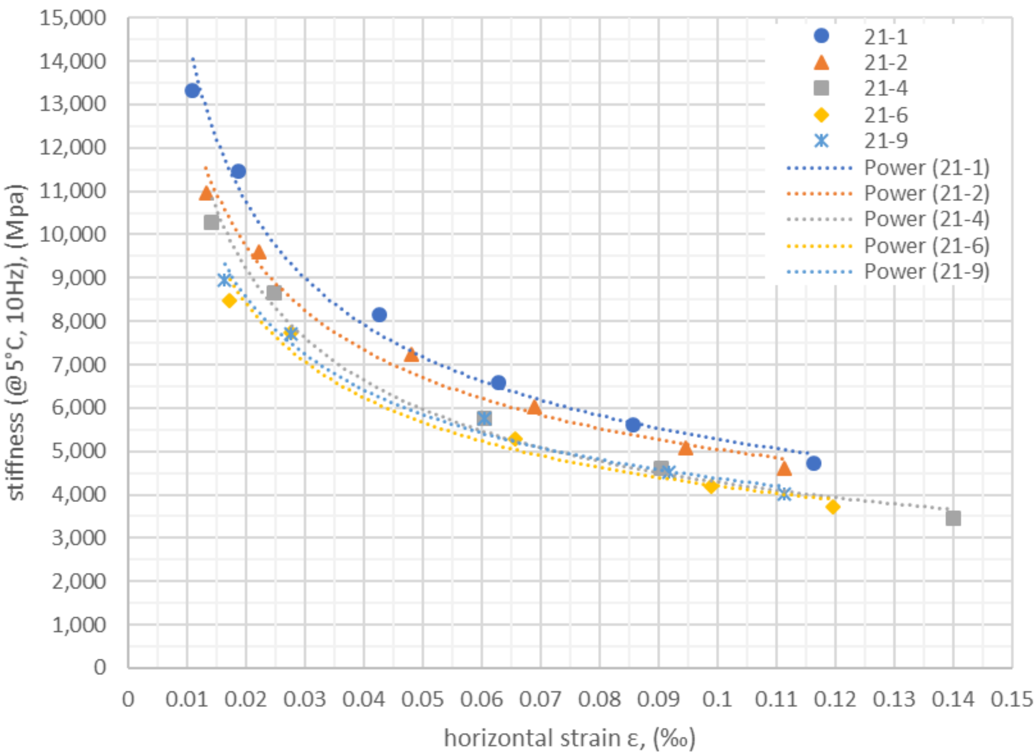


Figure 4-23: The effect of the test's strain level on the stiffness of the samples (F3.5C1.0, cured with the standard method) tested at 5 °C, 10 Hz.

Figure 4-23, shows the results for the 5 samples of the F3.5C1.0 mix (they are named based on the batch and sample number). It can be seen that the amount of stiffness decreases with the increase of horizontal strain level in the samples which shows that the material experiences some sort of damage in its microstructure (probably in the weakly cemented filler phase). The rate of change is higher in the strain ranges lower than 0.05‰ and decreases in the strain ranges from 0.05‰ to 0.1‰. These results confirm that if the stiffness is measured at the normal ranges for HMA, relatively lower amounts will be reported.

In reality, the amount of horizontal strain at the bottom of a layer of this material is normally higher than the low range which was selected for the stiffness tests. By knowing the relation between the strain level and stiffness, maybe it is possible to correct and adjust the stiffness for any other desired strain level. Table 4-8 shows the parameters of a power type equation fitted to the data points for each sample (parameter *a*, the constant and *b,* the power). It seems that the behavior follows the power equation very well (high amounts of R^2). The 3 lower samples had different densities, but they had the same stiffness in the range of 0.05‰ to 0.1‰ strain levels. The top two had higher densities and had a higher ranking in the stiffness results too. Considering the rankings on the strain range lower than 0.03‰ shows that the relation of density and stiffness is not unidirectional. Depending on which phase in the microstructure caused the difference in the density, the results will be different. By damage of that phase, the density will change and therefore, the measured density (which is measured at the no-load state) may have unlogical relation to the stiffness. This can be seen in the range of 0.05‰ to 0.1‰ strain levels. It is important to notice that this observation is not only because of the material's inherent characteristic but also the interaction of that with samples size and the test specifications (state of stress distribution in the indirect tensile test).

Table 4-8: The parameters of fitted power equation to the data of stiffness (in MPa) vs. horizontal strain level (in ‰) for F3.5C1.0 at 5 °C, 10 Hz

Power equation parameters, stiffness = a*(strain level)^b				
Sample ID	Constant (a)	Power (b)	R^2	Density (gr/cm^3)
21-1	1,906.5	-0.443	0.99	2.38
21-2	1,968.2	-0.409	0.98	2.37
21-4	1,435.6	-0.476	0.98	2.36
21-6	1,554.7	-0.432	0.98	2.35
21-9	1,672.0	-0.418	0.98	2.33

Based on the fitted equation to each sample's results, the stiffness was calculated at different strain levels from 0.025‰ to 0.1‰ for each of the samples. The coefficient of variations (CV%) between the stiffnesses of the samples at each strain level is 10% which is in an acceptable range. By having the stiffness at each point, average and standard deviation were determined and then the average stiffness at each strain level was determined with 95% and 99% levels of confidence (by using the table of student's t distribution with 4 degrees of freedom and one tail confidence level). Based on the amount of stiffnesses at each strain level, the differences between the two strain levels were calculated. Table 4-9 is made based on the mentioned results. The last two rows show the amount of stiffnesses with 95% and 99% of confidence (mentioned with 95% Con. and 99% Con.). Independent from the level of confidence, it can be seen that the amount of stiffness can differ up to 27% in the strain ranges of 0.05‰ to 0.1‰; the same difference can be seen in the lower ranges of 0.025‰ to 0.05‰ too.

Table 4-9: The stiffness difference between two different strain levels (F3.5C1.0, tests at 5 °C, 10 Hz)

Sample ID	Stiffness (MPa) at strain level (‰)				Difference (in%) between two strain levels				
	0.025	0.05	0.075	0.1	0.025 to 0.050	0.025 to 0.075	0.025 to 0.100	0.050 to 0.075	0.050 to 0.100
21-1	9,771	7,188	6,006	5,286	26	39	46	16	26
21-2	8,898	6,702	5,678	5,047	25	36	43	15	25
21-4	8,310	5,975	4,926	4,296	28	41	48	18	28
21-6	7,651	5,671	4,760	4,204	26	38	45	16	26
21-9	7,814	5,849	4,937	4,378	25	37	44	16	25
Ave.	8,489	6,277	5,261	4,642	26	38	45	16	26
95% Con.	7,663	5,664	4,740	4,175	26	38	46	16	26
99% Con.	7,038	5,200	4,345	3,820	26	38	46	16	27

The stiffness of the F3.5C1.0 mixes from the master curve at 5 °C and 10 Hz with asphaltic Poisson's ratio model is 7,154 MPa (see Table 4-7). The tests were performed at strain levels around 0.025‰. Comparing that with the model relative amounts at 95% and 99% confidence, (7,663 and 7,038 MPa, see Table 4-9) show differences of +7% and -1.6%, respectively. The parameters of 95% and 99% confidence equations are: a = 1,523.6; b = -0.438 and a = 1,387.5; b = -0.44 respectively. Considering that the samples of batch 21 (which were used for the model parameters) were produced and tested nearly one year after the ones used for the construction of the master curves, this level of agreement shows a good level of reproducibility of the samples and the tests' repeatability. It also shows that the constructed master curve for F3.5C1.0 (standard cured samples), has a good level of accuracy too.

Table 4-10, shows the predicted stiffness amounts by using these two equations and the initial stiffnesses from the fatigue tests on other samples with the same mix combination at the same strain level (fatigue tests on batch 3 samples). It shows a relatively good agreement between the test and the model considering that the model is based on the other set of samples than the tested ones.

Table 4-10: Comparing the initial stiffnesses from fatigue tests (F3.5C1.0 at 5 °C, 10 Hz) with the predicted ones from 95% and 99% level of confidence models

Sample ID	Strain level (‰)	Stiffness (MPa) with different methods			Difference with model (%)	
		Fatigue tests	95% Con. model	99% Con. model	95% Con. model	99% Con. model
3-6	0.082	4,228	4,556	4,170	7.8	-1.4
3-1	0.067	4,406	4,978	4,558	13.0	3.4
3-8	0.094	3,464	4,292	3,927	23.9	13.4
3-2	0.098	3,298	4,214	3,856	27.8	16.9

The same procedure was applied to the results of 6 samples of the F2.5C1.0 mix. Figure 4-24, shows the tests' results and Table 4-11, contains the parameters of the fitted power equations.

Table 4-11: The parameters of fitted power equation to the data of stiffness (MPa) vs. horizontal strain level (‰) for F2.5C1.0 at 5 °C, 10 Hz

Power equation parameters, stiffness = a*(strain level)^b				
Sample ID	Constant (a)	Power (b)	R^2	Density (gr/cm^3)
19-3	1,558	-0.427	0.97	2.40
19-4	1,371.5	-0.453	0.96	2.41
19-6	1,161.1	-0.486	0.98	2.40
19-9	1,374.5	-0.431	0.97	2.39
19-10	1,290.4	-0.448	0.97	2.39
19-11	1,539.3	-0.407	0.97	2.38

The parameters of 95% and 99% confidence level equations are: a = 1,277.7; b = -0.451 and a = 1,210.6; b = -0.458 respectively.

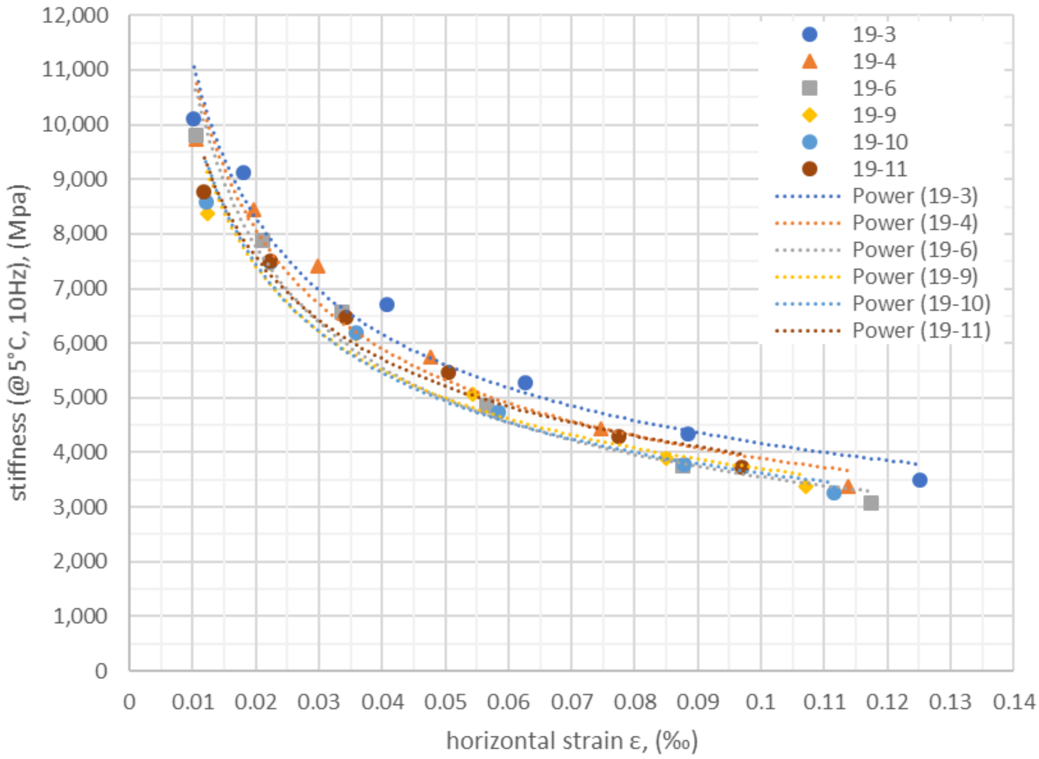


Figure 4-24: The effect of test's strain level on the stiffness of the samples (F2.5C1.0, cured with the standard method) tested at 5 °C, 10 Hz

Table 4-12, shows the stiffness difference between different strain levels. The coefficient of variations (CV%) between the stiffnesses of the samples at each strain level is 5% which is in an acceptable range.

Table 4-12: The stiffness difference between two different strain levels (from F2.5C1.0 stiffness tests performed at 5 °C, 10 Hz)

Sample ID	Stiffness (MPa) at strain level (‰)				Difference (in%) between two strain levels				
	0.025	0.05	0.075	0.1	0.025 to 0.050	0.025 to 0.075	0.025 to 0.100	0.050 to 0.075	0.050 to 0.100
19-3	7,527	5,599	4,709	4,165	26	37	45	16	26
19-4	7,293	5,328	4,434	3,892	27	39	47	17	27
19-6	6,974	4,979	4,089	3,555	29	41	49	18	29
19-9	6,740	4,999	4,198	3,708	26	38	45	16	26
19-10	6,737	4,938	4,118	3,620	27	39	46	17	27
19-11	6,908	5,210	4,417	3,929	25	36	43	15	25
Ave.	7,030	5,176	4,327	3,812	26	38	46	16	26
95% Con.	6,768	4,964	4,132	3,625	27	39	46	17	27
99% Con.	6,593	4,823	4,001	3,500	27	39	47	17	27

Independent from the level of confidence, it can be seen that the amount of stiffness can differ up to 27% in the range of 0.05‰ to 0.1‰; the same difference can be seen in the lower range of 0.025‰ to 0.05‰ too. Comparing these percentages with the ones for the F3.5C1.0 mix, it seems that in both mixes the amount of difference is the same around 27%. This shows that the strain level in the test, affects the resulted stiffness either in the lower range or in the higher range (normal range for HMA samples) almost the same but the amount of stiffness in the lower range is higher because of less damage in the sample.

The stiffness of the F2.5C1.0 mixes from the master curve at 5 °C and 10 Hz with asphaltic Poisson's ratio model is 7,381 MPa (see Table 4-7). The stiffness tests of that mix combination were performed at strain levels around 0.02‰. Comparing that with the relative amounts of 95% and 99% confidence levels, (7,481 and 7,322 MPa, based on the related equations) shows differences of +1.3% and -0.8%, respectively. Considering that the samples of batch 19 (used to develop the model parameters) were produced and tested nearly one year after the ones used for the construction of the master curves, this level of agreement shows a good level of reproducibility of the samples and the repeatability of the tests. It also shows that the constructed master curve for F2.5C1.0 (standard cured samples), has a good level of accuracy too.

As the second step, to assess if performing stiffness tests at relatively higher horizontal strain levels cause any internal damage in the samples, after multi-step stiffness tests, ITS tests were done on the same group of samples at 5 °C (named as group B). The remaining samples from each batch that were not tested in multi-step stiffness (the second group which was named as group A), were also tested for ITS at the same temperature for comparison. Table 4-14, shows the ITS amounts of the samples of each group for 2 mix combinations. Comparing the average ITS amounts of the two groups for each mix combination, It seems that the amount is not affected by performing the cyclic multi-step stiffness tests. So, it is possible to use the samples after cyclic stiffness tests for the ITS test too.

Table 4-13: Effect of performing stiffness tests on the ITS results (group A, only ITS tests and group B, ITS tests after multi-step cyclic stiffness tests)

Mix	ITS@5 °C (kPa) of group A					ITS@5 °C (kPa) of group B				
F3.5C1.0	875	929	784	773	794	869	875	824	804	749
	Average		831			Average		824		
Mix	ITS@5 °C (kPa) of group A				ITS@5 °C (kPa) of group B					
F2.5C1.0	712	720	686	633	759	626	663	684	706	704
	Average		688		Average		690			

For a more detailed assessment, the load-deformation responses of the samples were considered by comparing the shape of vertical load vs. horizontal deformation graph (which has the same shape as the vertical stress vs. horizontal strain graph) during the ITS tests. Figure 4-25 and Figure 4-26 show these graphs for F2.5C1.0 and F3.5C1.0 mix combinations.

Comparing the A and B groups' graphs in both mix combinations shows that independent of the amount of bitumen, the group B graphs are more linear in the first part of the test but they have higher horizontal deformation than group A, at the same vertical load; which means that they have lower stiffness. It seems that a type of damage has formed in the samples due to the multi-step stiffness tests. To clarify that, the material's microstructure model shall be discussed; During the multi-step stiffness tests increasing the vertical load to achieve higher horizontal strain levels, leads to the damage of the weaker phases (free filler and lightly cemented filler phases) and the decrease of stiffness.

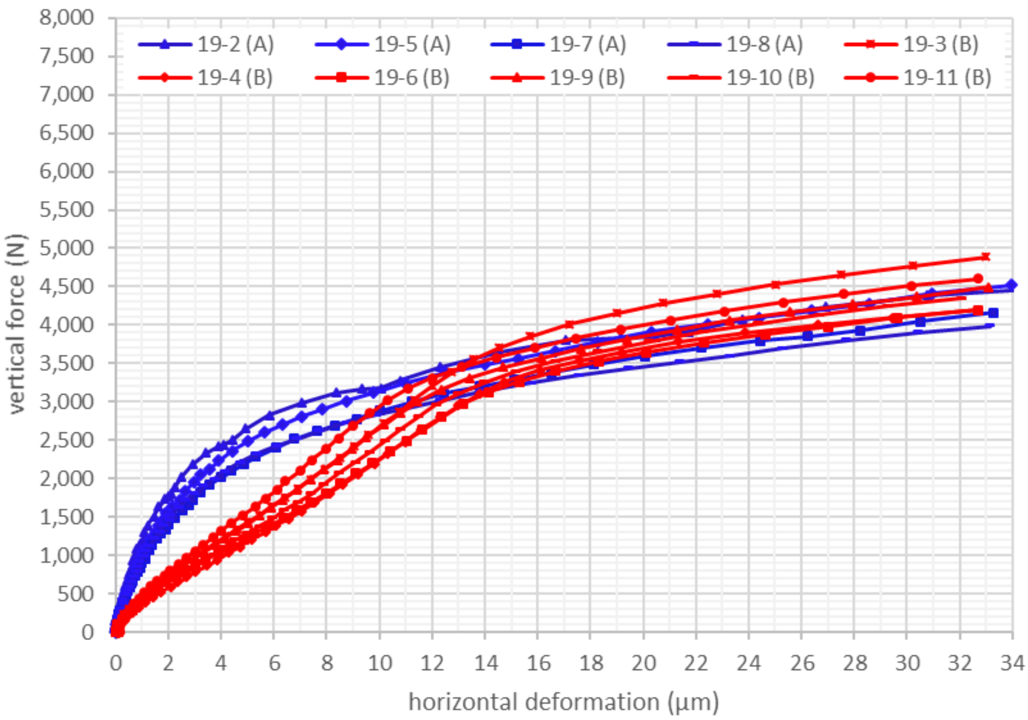


Figure 4-25: load-deformation behavior of group A (directly performed ITS) and group B (ITS performed after cyclic stiffness tests) samples during ITS tests @ 5 °C, F2.5C1.0 mix

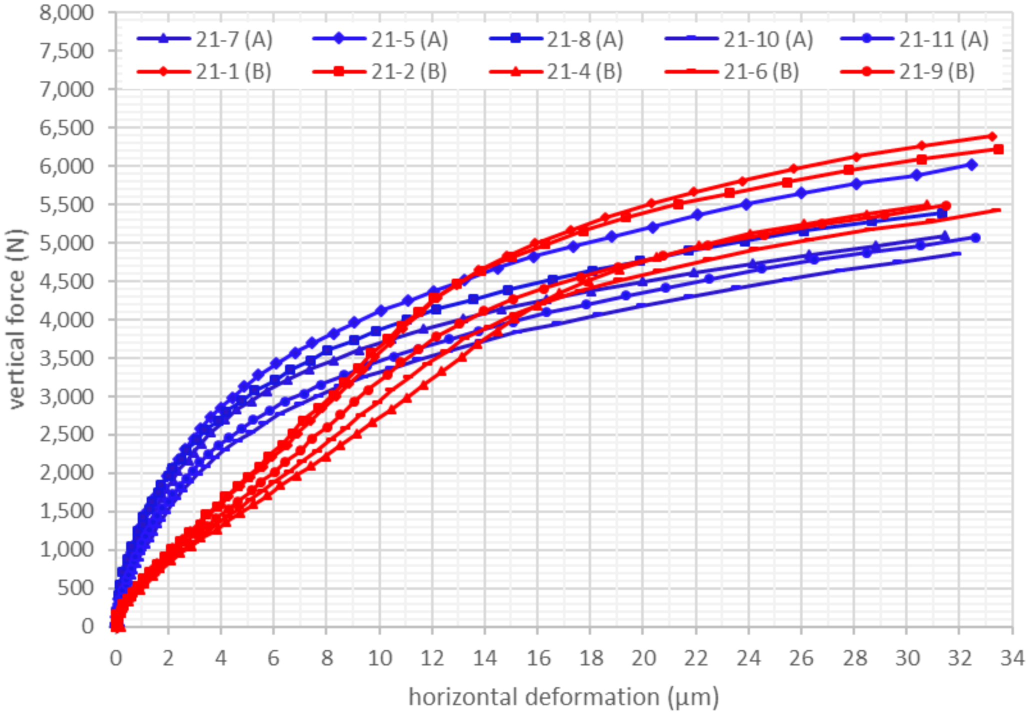


Figure 4-26: load-deformation behavior of group A (directly performed ITS) and group B (ITS performed after cyclic stiffness tests) samples during ITS tests @ 5 °C, F3.5C1.0 mix

Considering the above-mentioned point, the damaged filler phase has different strength characteristics than the undamaged one. This new phase is still the weaker one compared to the other phases, therefore the first part of the graphs has changed (having a lower slope compared to the group A graphs) but as the bituminous mastic phase has remained intact, so the graph continuous the old trend with the progress of the test and increase of the load. Back to the graphs, it is important to notice that depending on the selected level of load and deformation (for calculation of the elastic modulus from the ITS test), different amounts will result when comparing the group, A and B samples. Therefore, if the samples were already used for the cyclic stiffness tests at relatively high horizontal strain levels, using them to determine the elastic modulus from the ITS test should be done with caution.

To see if this argument about the weakness of the filler phase for the mixtures with 1% cement is right or not, 4 samples with the same gradation, compaction method and curing procedure as the above-mentioned samples were prepared; with only 2% cement and no bitumen (F0.0C2.0). Two were used for the multi-step stiffness (like the ones before) and then the ITS test (group B) and 2 were used directly for ITS (like group A).

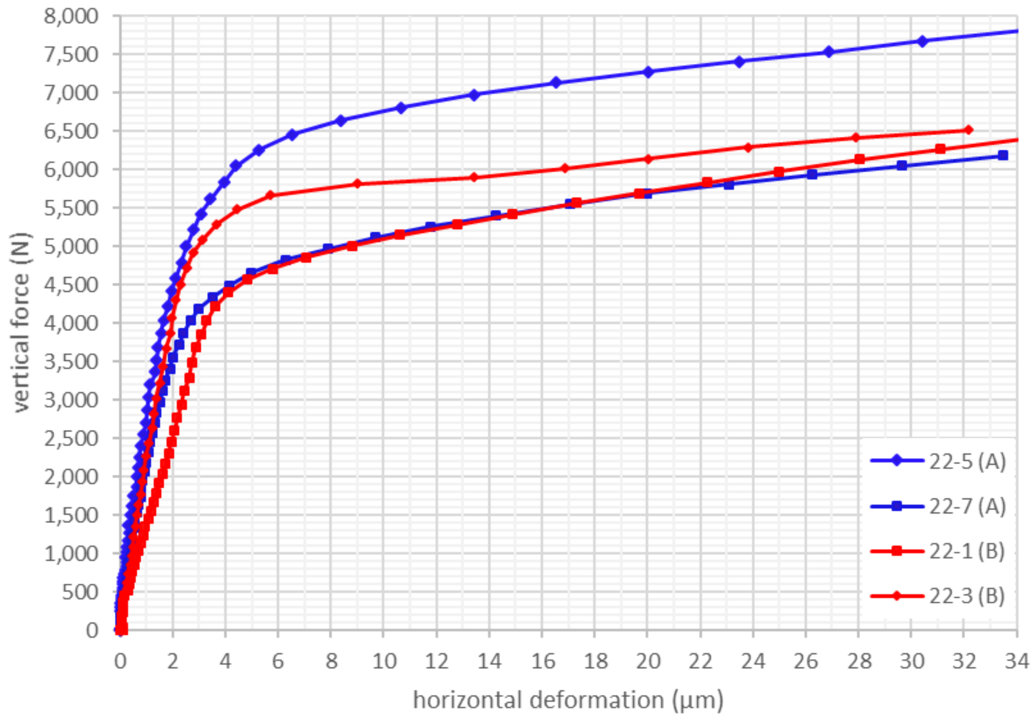


Figure 4-27: load-deformation behavior of group A (directly performed ITS) and group B (ITS performed after cyclic stiffness tests) samples during ITS tests @ 20 °C, F0.0C2.0 mix

Figure 4-27, shows the graphs of group A and B samples. it can be seen that they have almost the same trend and behavior. This shows that what was damaged was the weak filler phase and supports the above-mentioned argument as the 1% cement doesn't lead to the formation of a uniform cemented phase from the free filler phase.

Gonzalez et al. [41] made a conceptual model to interpret the damage steps during the ITS test of foamed bitumen mixes based on the material's microstructure introduced by Fu (see Figure 2-2). The model is a simple system consisting of springs that represent the cemented filler and the bituminous mastic phases (figure 6 of the reference [41]). Based on their model, the inclusion of bitumen will decrease the slope of the stress-strain line. The above-presented results of samples with bitumen and cement and with only cement, confirm their conceptual model.

4.8.2 Multi-round stiffness tests

After assessing the effect of strain level on stiffness in a constant temperature, the next step is to see the effect of strain history on the stiffness and how the temperature dependency of stiffness changes with the strain level (or with damage level). To figure out this question, different rounds of the multi-step stiffness tests were performed on F3.5C1.0 mix combination samples (cured with the fast curing procedure) at different temperatures (20, 5, 0 and -10 °C) and frequency of 10 Hz. As the aim was to see if temperature dependency changes with strain level, only one sample was tested at each temperature (except at 20 °C, that 3 samples were tested). The first round was performed by increasing the strain at each step (by increasing the stress amount for each stiffness test) up to around 0.1‰ for each sample, to see the effect of damage on the material's stiffness behavior. After that, the second round of multi-step stiffness tests were done on the same samples (starting from a low strain up to 0.1‰). One of the samples at 20 °C temperature was stressed up to a lower strain level (around 0.05‰) and for three rounds to see if the experienced maximum strain at the first round has any effect on the behavior in the next rounds. As in this test, the multi-step stiffness tests are performed in more than one round, it is named as multi-round stiffness test. The idea behind this test is to see how the material behaves after it is damaged. Asphaltic model for Poisson's ratio was used for calculation of stiffnesses at each step. The selection of 0.05‰ and 0.1‰ as higher limits was because they are the upper and lower ranges for indirect cyclic tests of hot mix asphalt and are utilized by some researchers.

Figure 4-28 shows the results for different samples at different temperatures. The first round results have the same trend as the ones for F2.5C1.0 and F3.5C1.0 mix samples at 5 °C. Comparing different temperature graphs shows that stiffness's temperature dependency still exists. Considering second-round results, the stiffness became almost constant with changing the test's strain level. It seems that the first round caused a state of damage and reorientation in the microstructure therefore, during the second round a kind of resilience response has happened. The amount is almost the same as the final amount in the first round. This behavior is like what happens in granular material which the stiffness becomes resilient after some load reparation.

Looking at the graphs of the samples that were tested at 20 °C (Figure 4-28), the one which experienced a lower amount of strain at the first round (sample 17-4), had a higher difference between the results of the first and second round, but in the third round, the difference became smaller. It shows that the amount of resilient stiffness is dependent on the highest level of strain or better to say, the level of damage that the material has already experienced. It means that the material will damage by increasing the load (results in an increase in the horizontal strain) but after repeating the load, a state of resiliency is formed, and the stiffness becomes constant. This state is valid up to the maximum strain level that the material has already experienced and if a further higher amount is put on the material, the stiffness will decrease again and a new state of equilibrium with a new amount of stiffness will occur. The point is that the amount of resilience stiffness is almost the same as the amount of stiffness at the highest strain level in the first round. It shows that the strain (stress) history can affect the stiffness of the material. These observations can be utilized during the selection of material's stiffness for pavement modeling and structural design purposes.

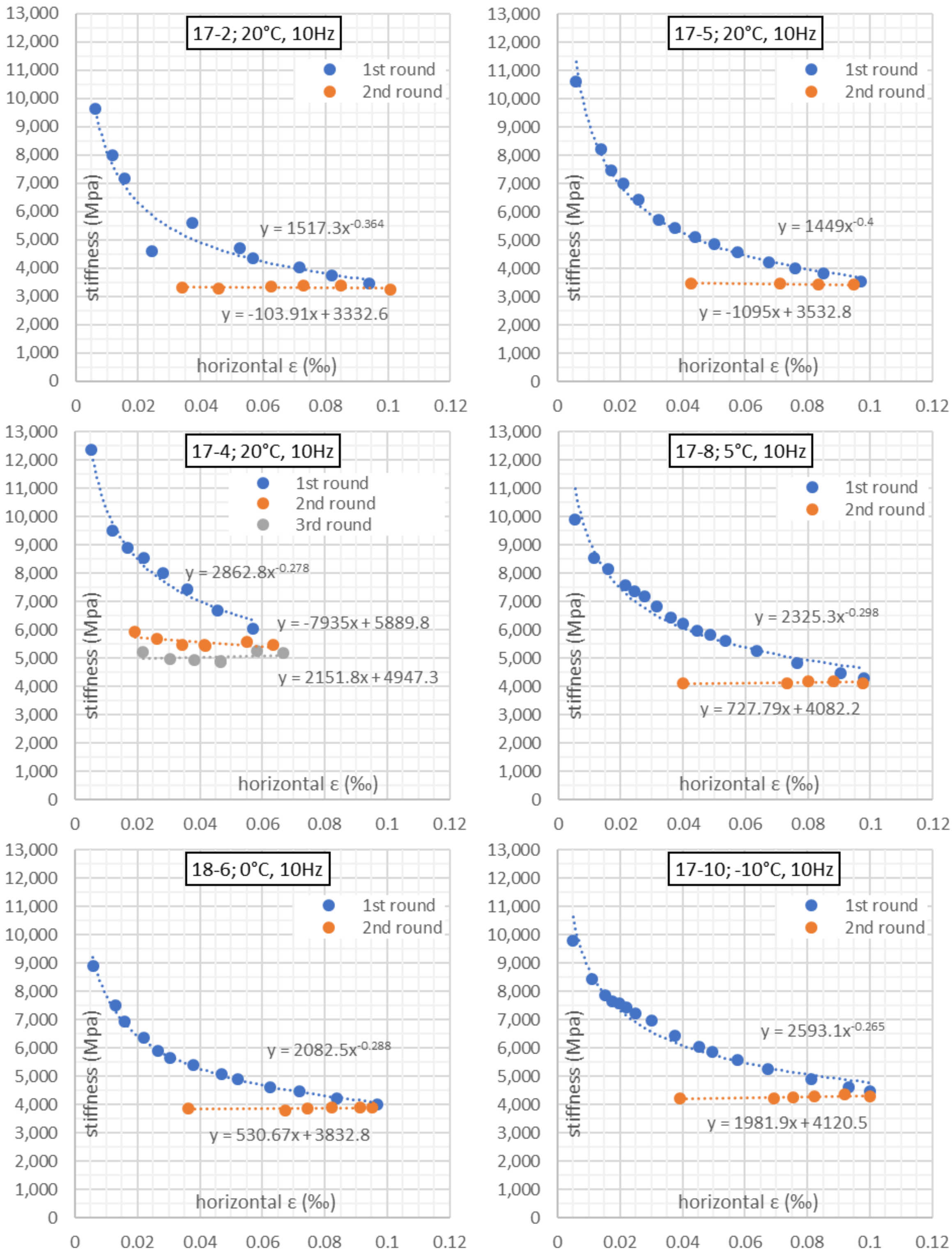


Figure 4-28: The effect of test's strain level on the stiffness of the samples (F3.5C1.0, cured with the fast method) tested at different temperatures, 10 Hz

To get a better impression, stiffness amounts were calculated at different strains from the fitted equations to the results. Table 4-14 shows the amounts; comparing the difference of stiffness between two strain levels, shows that in the second round the effect of the test's strain level is almost negligible.

Table 4-14: The effect of test temperature on the stiffness at different strain levels (F3.5C1.0, cured with the fast method) tested at 10 Hz

Sample ID and test's Temperature	Stiffness at strain level/round						Difference between the two strain levels (in%)			
	0.025‰		0.05‰		0.1‰		0.025 / 0.05		0.050 / 0.1	
	1st	2nd	1st	2nd	1st	2nd	1st	2nd	1st	2nd
17-2; 20°C	5,811	3,330	4,515	3,327	3,508	3,322	22	0.1	22	0.2
17-5; 20°C	6,337	3,505	4,803	3,478	3,640	3,423	24	0.8	24	1.6
17-8; 5°C	6,981	4,100	5,678	4,119	4,618	4,155	19	0.4	19	0.9
18-6; 0°C	6,025	3,846	4,935	3,859	4,042	3,886	18	0.3	18	0.7
17-10; -10°C	6,892	4,170	5,736	4,220	4,773	4,319	17	1.2	17	2.3

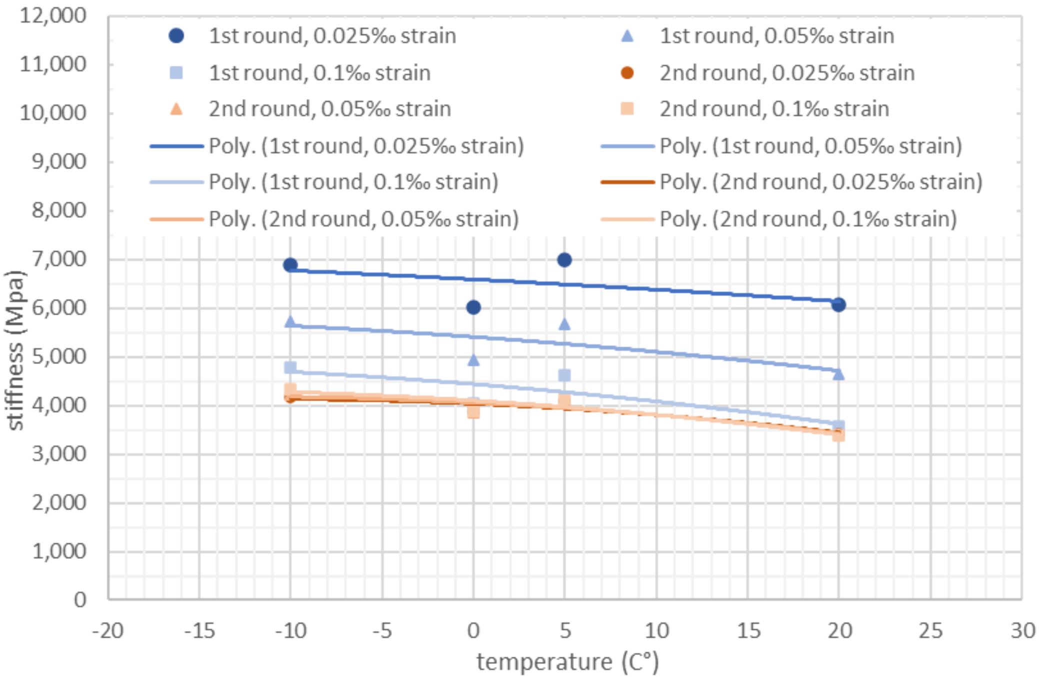


Figure 4-29: Temperature dependency of the stiffness measured at different testing horizontal strain levels and different loading rounds (F3.5C1.0, cured with fast method, tested at 10 Hz)

Figure 4-29 shows the temperature dependency of the stiffness at different strain levels and different test rounds. Considering the first round's graphs, the temperature sensitivity has a small increase with the test's strain level. By strain increase, the material starts to experience micro damages in areas with the weaker cemented filler phase; this will affect its temperature sensitivity (more in tensile mode than the compressive mode). To prove this hypothesis, more samples should be tested. It is possible to say that the temperature sensitivity doesn't change with the level of the horizontal strain during the test (at ranges up to 0.1‰).

The second round shows the sample state after experiencing a specific level of damage; the material is still temperature dependent, but the amount of stiffness is independent of the test's strain level. It seems that the temperature dependency of stiffness decreases with temperature and is lower at lower temperature ranges. Loading the samples to reach the 0.1‰ horizontal strain, will cause more damage at lower temperatures which results in lower stiffness amounts and results in the fall of the stiffness-temperature graph in the lower temperature region. The second point is that the damaged material may have another Poisson's ratio than the undamaged one at different temperatures. It is important to mention that this behavior is valid as long as the new loading doesn't produce strains higher than the maximum experienced in the previous round.

To investigate more, a series of samples (batch 24 with F3.5C1.0 mix combination, standard cured) were preloaded (at 20 °C and 10 Hz) to reach the horizontal strains around 0.08‰. This preloading is like the first round of loading in the multi-round tests but as it is at the same temperature for all the samples, the damage level may be the same for all. then, they were tested at different temperatures and frequencies to construct the master curve (the tests were performed at a horizontal strain range of 0.07 to 0.08‰).

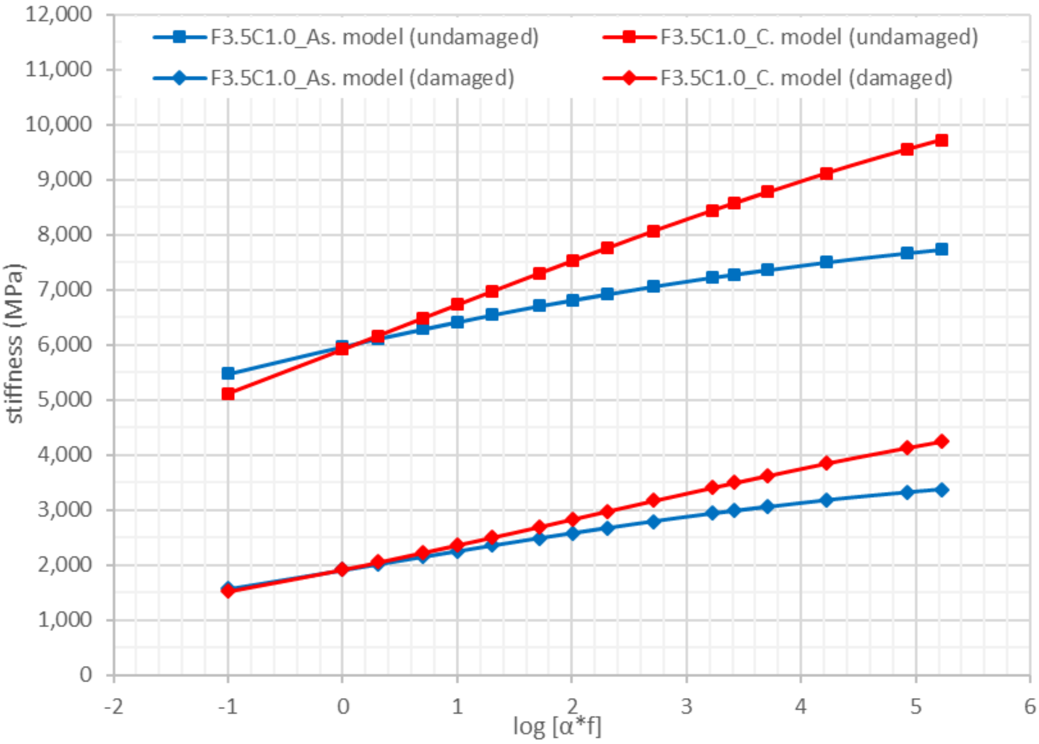


Figure 4-30: Comparing the undamaged and damaged state master curves of F3.5C1.0 samples with two different Poisson's ratio models (As. as asphaltic and C. as constant)

Figure 4-30, shows the constructed master curves from the mentioned test results (named as damaged) with two different Poisson's ratio models (asphaltic and constant) and the master curves of the same mix combination (presented before in Figure 4-20) in an undamaged state. It can be seen from the damaged state master curves that the material still has temperature dependency independent of Poisson's ratio model; as concluded before. It also shows that the resulting master curve is still affected by Poisson's ratio model like before in the case of undamaged samples, but the effect has decreased. Looking at the master curve in a damaged state confirms the above-mentioned argument on how the resulted master curve can differ with the damage level of the samples and the importance of having a similar level of damage in all of the samples which are used in a stiffness-temperature test round.

Considering all of these results and observations reveals how the test's specifications and boundary conditions can affect the calculated stiffness amounts of the material which may even lead to wrong interpretations from its mechanical behavior too.

4.9 Effect of the loading orientation on the stiffness results

Cylindrical samples of multi-phase materials like HMA or FCSM, have variable aggregates' distribution and orientation in different diametrical planes. The effect of this heterogeneity on indirect tensile tests results has been investigated by different tests and numerical simulations for the HMA material [195, 196]. According to Wang & Hao, component phases of HMA are heterogeneous and are non homogeneously dispersed. The size, amount of aggregates, and mastic distribution are not uniform through the loading direction; resulting in different stress and strain states in the mix and influencing the resulting stiffness. They suggested using multiple specimens for the tests to reduce this effect [196]. Figure 4-31 shows an example of the stress distribution along the loading axis of the specimen, taken from their simulation results. It shows how far the real distribution can be different from the ideal assumed one.

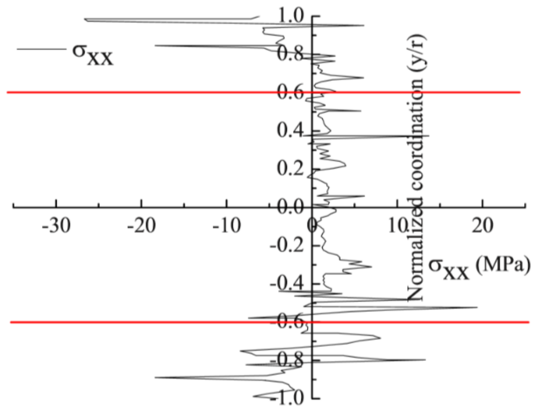


Figure 4-31: Stress distribution along the vertical axis of the sample in indirect tensile test [196]

As FCSMs are also multi-phase component mixtures with the inclusion of cemented or free filler phase, the heterogeneity of the mixture will be more in different diametral directions; therefore, it is expected that the loading orientation to affect the resulted stiffness and also the position of microdamage initiation points in the weaker phases. As the laboratory specimens of the HMA samples in Germany are normally cored from the compacted slabs, loading direction is determined based on the compaction direction; so, some kind of standardization can be made. When other compaction methods (Marshall, Mod. Proctor, vibratory hammer or gyratory) are used, it is not possible to define a standard unified direction. According to EN12697-26:2012 (D) for HMA cylindrical samples, two perpendicular axis (90° ± 10°) are tested, if the result of the 2nd direction lies between 110% and 80% of the 1st direction, the mean of the two directions will be taken as the stiffness. If not, the results will be rejected. Considering the strain level dependency of the stiffness in FCMs, applying this method should be done with caution.

Table 4-15 shows the results of the stiffness tests on 3 samples (F3.5C1.0 mix, fast cured) performed at 2 different loading directions (90° apart) at 20 °C and 10 Hz. If performing the test in the first direction leads to internal damage, then the test's results in the other directions are affected and are not reflecting the undamaged state of that direction. To avoid this, the tests were performed on a low strain range. Later based on the findings of the last section, knowing that the relation of the stiffness vs. strain level is a power equation, the stiffnesses on higher strains were determined with the regression method. It means that second direction stiffnesses are not affected by the first direction damage but, the damage in each direction is considered in the results. Make it simple, the results model the situation when a user selects one direction accidentally for the testing.

Table 4-15: The effect of loading direction on the stiffness at different strain levels (F3.5C1.0, cured with the fast method) tested at 10 Hz

Sample ID, direction	Stiffness (MPa) at strain level (‰)				Difference between two directions (%)			
	0.025	0.05	0.075	0.1	0.025	0.05	0.075	0.1
17-6, 1st	7,077	5,653	4,957	4,516	14	12	10	9
17-6, 2nd	6,071	4,993	4,453	4,106				
17-3, 1st	6,161	4,649	3,944	3,509	-25	-28	-30	-32
17-3, 2nd	7,673	5,958	5,138	4,626				
17-1, 1st	7,124	5,497	4,723	4,242	-24	-18	-14	-11
17-1, 2nd	8,849	6,469	5,385	4,729				
Difference between Min. and Max. stiffness (%)					-46	-39	-37	-35

Table 4-15 is only an example to show the effect of loading orientation on the results. It can be seen that the difference between the two directions can be variable and up to 32%. The last raw shows the difference between the minimum and maximum amounts between all results. They reflect the worst situation from different combination's possibilities of these 3 samples' results. It seems that the difference decreases with the increase of the test's strain level. This observation agrees with the findings of the previous section and can be because the effect of the free or weakly cemented filler phases will become less at higher strain levels.

To reduce the effects of variations in aggregates and other phases (bituminous mastic and filler phase) distributions, multiple tests on different samples are suggested. It is also important to select the appropriate sample sizes based on the maximum size of the aggregates in the mix.

4.10 Static vs. Dynamic tests for stiffness determination

The response of viscoelastic mixtures is different regarding the load rate and temperature. Therefore, the load type can affect the resulting stiffness. Stiffness master curves showed that the FCSM is noticeably lower temperature and load rate dependent compared to HMA even with the same mix design. Based on the M KRC guide [11], the results of the indirect tensile strength test at 5 °C can be used to calculate the elastic modulus of the mixture. It suggests the Poisson's ratio of 0.3 and the load and related horizontal deformation at 45% of maximum load (failure load) for calculations. The mix will be considered as bituminous dominant when the calculated elastic modulus is between 3,000 to 7,000 MPa [11]. To see if this range is valid for the mixes of this research, the ITS tests data of two mix combinations of F3.5C1.0 (batch 21) and F2.5C1.0 (batch 19) which were performed at 5 °C were used to calculate the modulus and the strain at break. For each mix combination, 2 groups of samples were available. Group A, the samples that had the ITS test at 5 °C (with a loading increase rate of 50 mm/Min.) and group B, the samples that were used first for multi-step cyclic stiffness tests at 5 °C and 10 Hz (at different strains up to 0.12‰, see section 4.8.1), then ITS tests in the same temperature.

Table 4-16 shows the calculated elastic modulus (E_{sz}) and strain at break (ε_{sz}) for the F3.5C1.0 samples. The elastic modulus of group A samples shows a good agreement with the M KRC lower range. Comparing the results of the two groups together confirms that applying the horizontal strain ranges of HMA for FCSMs leads to damage in the samples with a low amount of cement (≤ 1%) as the cement amount seems to be not enough to form a relatively bond filler phase.

Table 4-16: Elastic modulus (E_{sz}) and strain at break (ε_{sz}) of F3.5C1.0 samples calculated based on M KRC at 5 °C (group A, directly ITS and group B, ITS after indirect tensile cyclic multi-step stiffness test)

Group A samples			Group B samples		
Sample ID	E_{sz} (MPa)	ε_{sz} (‰)	Sample ID	E_{sz} (MPa)	ε_{sz} (‰)
21-7	3,414	18.407	21-1	3,405	6.141
21-5	4,068	7.333	21-2	3,464	8.107
21-8	4,617	6.913	21-4	2,434	9.515
21-10	3,027	8.875	21-6	2,732	6.690
21-11	3,184	8.936	21-9	2,989	7.707
Average E_{sz}	3,662		Average E_{sz}	3,005	

Looking at the amounts of strain at break, they are in the range of 6 to 9.5‰, except the one for sample 21-7. The same has been already observed between the results of the two samples with the F3.5C1.0 mix combination at 20 °C (see Figure 4-14). Figure 4-32, shows the graph of vertical force vs. horizontal deformation made from ITS tests data of the samples.

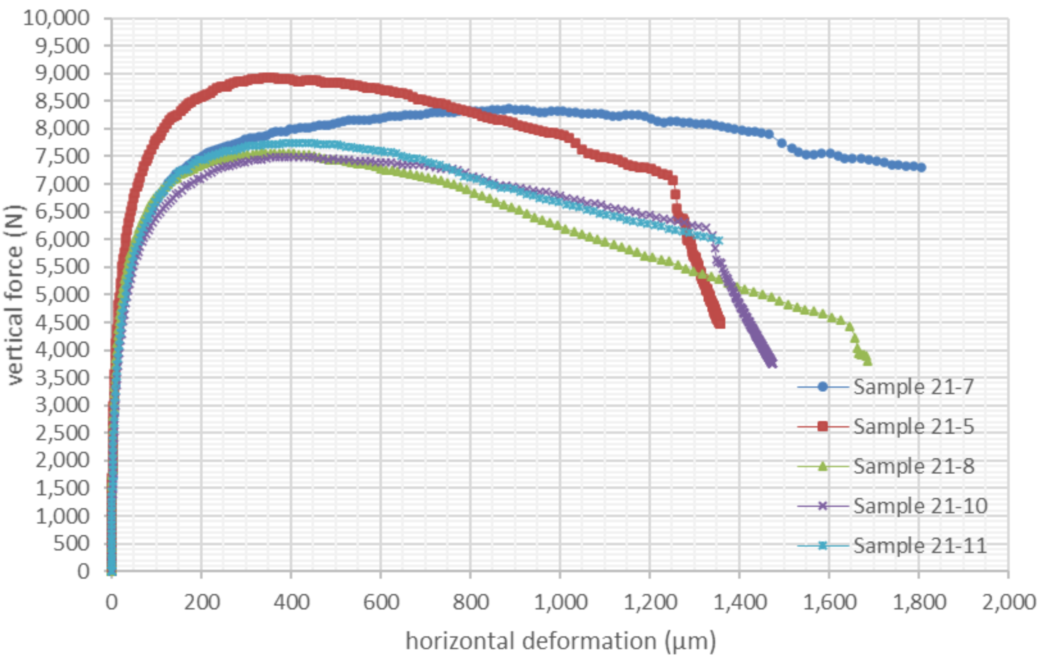


Figure 4-32: Vertical force versus horizontal deformation graph from ITS tests (F3.5C1.0 tested at 5 °C)

Comparing the shape of the graphs together, sample 21-7 is noticeably different from the rest and is flatter before and after the failure area. As this observation is one in the group of 10 results, it can be because of the sample's heterogeneity. On the other side, as it may also happen in the case of 2 results, it is recommended to test enough samples and also always look at the graph shape too. As the second solution, in cases that this index is used to compare the rigidity of different mix combinations, the author recommends using the horizontal deformation related to 90 or 95% of maximum force (instead of the 100% point) to calculate the horizontal strain. It is not exactly the strain at break but as mentioned before, can be used as a comparing index.

Table 4-17 shows the elastic modulus and the strain at break for the F2.5C1.0 samples, the same as the F3.5C1.0 mixes, the cyclic stiffness tests seem to cause damages in the samples. The average elastic modulus of group A samples is in the lower range of M KRC (3,000 MPa). Looking at the ITS amounts of these mixes (see Table 4-13), with the average of 689 KPa, shows that they are lower than the minimum acceptable limit of M KRC (which is 750 KPa for the 28 days samples at 5 °C). It shows that the ITS and elastic modulus criteria mentioned in M KRC have a good agreement together. Comparing the average results of elastic modulus of the two groups confirms that applying the horizontal strain ranges of HMA for FCSMs leads to damage in the samples and results in a lower amount of elastic modulus in group B samples.

Table 4-17: Elastic modulus (E_{sz}) and strain at break (ε_{sz}) of F2.5C1.0 samples calculated based on M KRC at 5 °C (group A, directly ITS and group B, ITS after indirect tensile cyclic stiffness test)

Group A samples			Group B samples		
Sample ID	E_{sz} (MPa)	ε_{sz} (‰)	Sample ID	E_{sz} (MPa)	ε_{sz} (‰)
19-2	3,908	13.507	19-3	2,575	10.161
19-5	3,104	6.610	19-4	2,214	11.904
19-7	2,679	9.734	19-6	2,194	6.622
19-8	3,051	8.170	19-9	2,532	7.229
Average E_{sz}	3,186		19-10	2,320	8.646
			19-11	2,808	8.253
			Average E_{sz}	2,441	

Comparing the elastic modulus of the two mix combinations of F3.5C1.0 and F2.5C1.0 (group A samples from Table 4-16 and Table 4-17) with M KRC criteria shows that F3.5C1.0 is in the area of bitumen dominant bond material but F2.5C1.0 is on the marginal zone and can’t be considered in the group of bond behavior mixes. Regarding the author’s experiences and international research findings, both conclusions are right. The F2.5C1.0 mix contains low amounts of bitumen and cement and can be classified as the South African BSM which is considered a semi-bond mix. It seems that the M KRC criteria are applicable for the mixes with bonded behavior and they were grounded based on this intention too.

To compare the elastic modulus calculated from the ITS test with the dynamic stiffness modulus determined from a cyclic indirect tensile test, the results of tests on two groups of samples from batch 21 with F3.5C1.0 mix combination (named as A and B), were used for this analysis. The amount of Poisson’s ratio was selected 0.28 for both groups as calculated before from the ITS tests’ data. The dynamic stiffness of group B samples was recalculated at different strain levels with 0.28 as Poisson’s ratio. For group A, instead of using only the horizontal deformation and related load at 45% of the maximum point, different levels from 15% to 45% were used. After calculation of the elastic modulus at each level, it was used to calculate the horizontal strain at that level too. By this approach, it is possible to have the elastic modulus from ITS tests data at different strain levels with better comparison possibility of the two test methods results.

Figure 4-33 shows the stiffness results of the two test methods at different tensile stress levels. Besides the scatter of the results, it seems that the load type has a low affect on the response of the material at 5 °C. Noticing low temperature and loading rate dependency of stiffness in FCSMs explains these results.

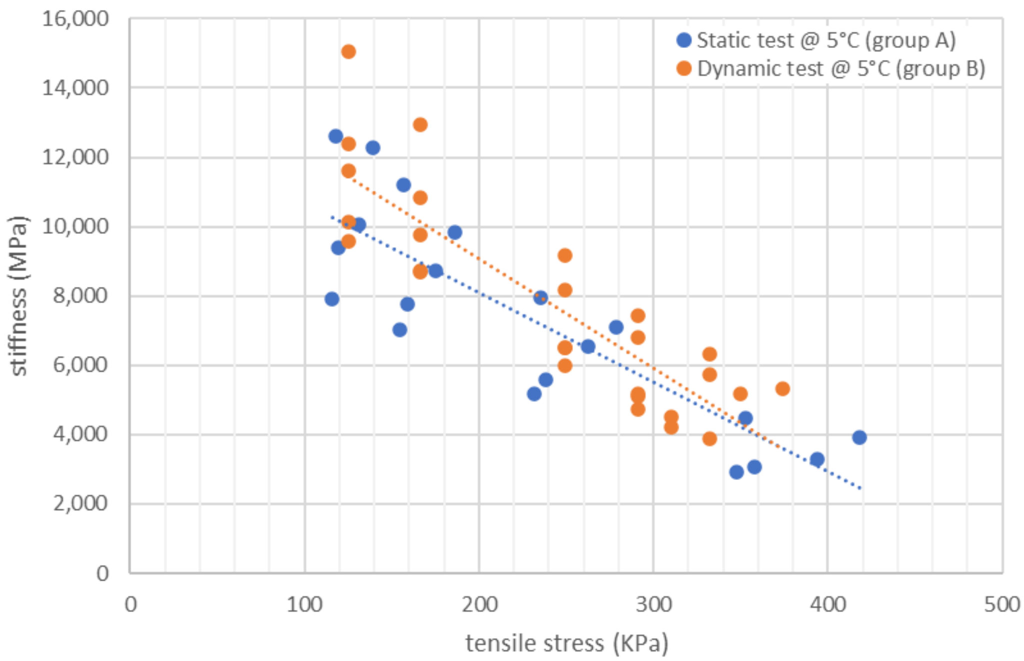


Figure 4-33: Comparing the stiffnesses from static (50 mm/Min., 5 °C) and cyclic (10 Hz, 5 °C) indirect tensile tests at different stress levels (F3.5C1.0)

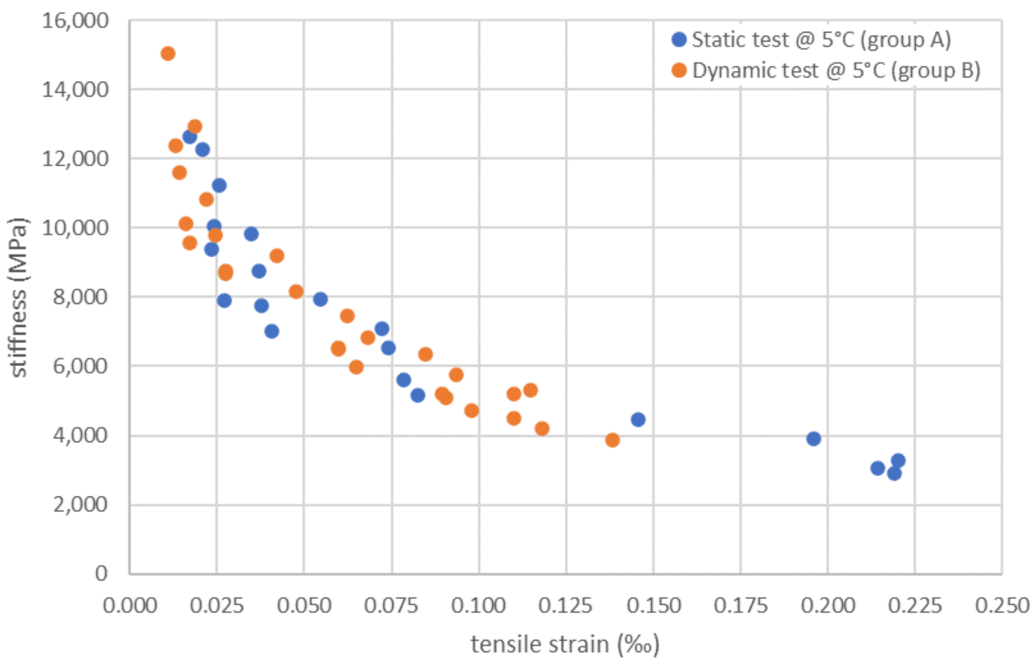


Figure 4-34: Comparing the stiffnesses from static (50 mm/Min., 5 °C) and cyclic (10 Hz, 5 °C) indirect tensile tests at different strain levels (F3.5C1.0)

Figure 4-34, shows the comparison based on the test's level of horizontal strain. Again, it seems that the results are almost the same. It is important to mention that a static load that increases with displacement rate control, affects the micro-structure in a different way than a cyclic load type, especially the free / weakly cemented filler phase. Therefore, based on these observations it is not possible to conclude that the static test can be used instead of the cyclic test. More tests and samples at different temperatures and frequencies are needed to be tested too. The only conclusion is that these observations also confirm the lower load rate dependency of FCSMs in comparison with HMA.

4.11 Fatigue behavior of the foamed bitumen-cement mixtures

The fatigue tests on specimens with different mix combinations aiming to assess the effect of binding agents were performed at 5 °C and 10 Hz. The main reason for this decision was the idea that during an indirect tensile fatigue test at intermediate temperatures on a visco-elastic material, a combination of fatigue cracking and plastic deformation occurs therefore to capture the fatigue failure, a lower temperature is maybe better. Using 5 °C for cold bituminous mixes was also recommended by Twagira based on fatigue tests on these types of mixes [30].

later the effect of temperature was assessed by performing tests at different temperatures. As confirmed by the results of the multi-step stiffness tests, because of the sample damage, it is not appropriate to perform a multi-step stiffness test on a sample to determine the needed stress ranges for different initial strain levels. Considering this and the limited number of samples for each mix design, it was decided to use the data from ITS tests and select the first stress level for the first sample's fatigue test and then based on the results of each test, select the next one's stress level aiming to sweep a wide and acceptable range of strains as much as possible. In the case of higher cement contents, the mixes were more sensitive to the test's stress level and it wasn't easy to cover a wide range of strains.

One of the challenges with the fatigue tests is the definition of the fatigue point to determine the laboratory fatigue life. In this research, the dissipated energy ratio was the main method to define the fatigue point. In this method, the energy ratio (ER) of each cycle versus the loading cycle is depicted in a graph. The ER amount increases with the loading cycles but the rate decreases as the micro-damages initiate and grow in the sample with the load repetitions, till a maximum point and then decreases with the loading cycles till the sample breaks. The data points around the maximum area are used to determine the maximum ER point which is the point of macro-damage and is taken as the fatigue point. The calculation method is explained in AL Sp-Asphalt 09 [107] or the TP Asphalt-StB, Teil 24 [188]. Figure 4-35 shows an example of the ER graph of a fatigue test on a specimen with F3.5C1.0 mix combination, performed at 5 °C under cyclic load with a loading rate of 10 Hz.

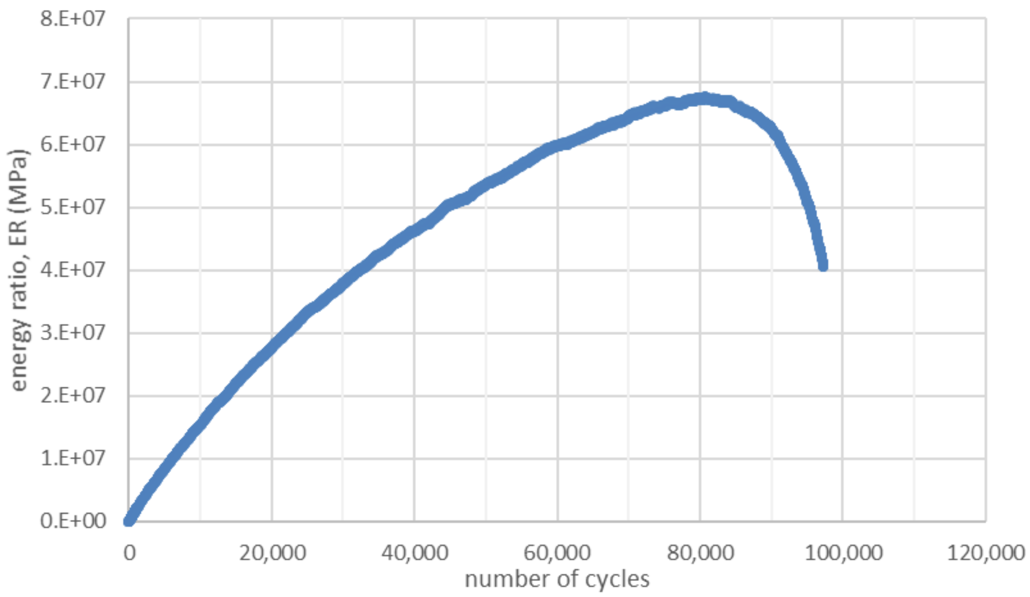


Figure 4-35: An example of energy ratio graph (ER graph) from fatigue tests on F3.5C1.0 mixes performed at 5 °C, 10 Hz (sample 3-7)

The method works relatively good in continuously bonded materials like HMA but in the case of non-continuous and multi-phase mixtures like FCSMs, the interaction of material's complexity with the state of stress distribution (because of the test method) affects the damage growth from micro-damage points to the main crack in a different way. This may be reflected in the ER graphs and change their shape. Figure 4-36 is an example of a strange ER graph which can be explained with the above-mentioned points. The amount of these results was very low, therefore, when it is not possible to define the fatigue life in such cases, the fatigue equation can be determined without considering them.

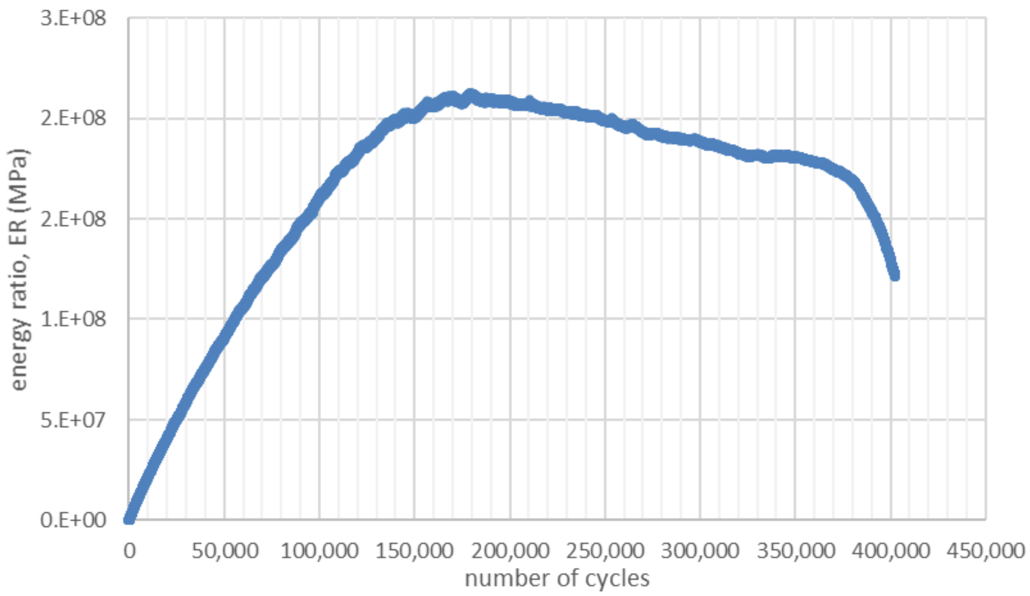


Figure 4-36: Unusual ER graph (sample 3-10 with F3.5C1.0 mix performed at 5 °C, 10 Hz)

Besides the ER graph, there are two other graph types that can show the changes during this type of fatigue test. The stiffness vs. the number of load cycles (which is somehow the base of the ER graph too) and the cumulative horizontal plastic deformation vs. load cycles.

Figure 4-37 shows an example of a typical graph of stiffness vs. load cycles. The stiffness has a sharp decrease with the increase of load cycles at the first zone of the graph, then the rate decreases considerably and becomes relatively constant (second zone). This trend continues for a while till a point that it decreases again sharply (third zone) which is the region of the sample's break. The same shape was from 4-point beam fatigue tests on foam mixes at 5 °C and 10 Hz reported by Twagira (page 72) [30],.

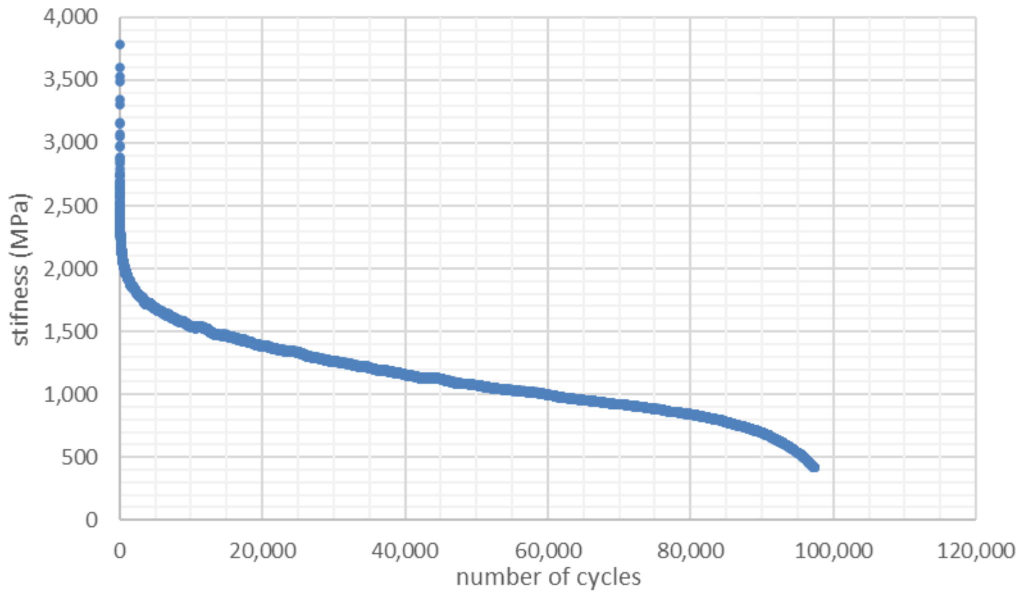


Figure 4-37: An example of stiffness change during the fatigue test on a sample of F3.5C1.0 mix, performed at 5 °C, 10 Hz (sample 3-7)

Figure 4-38 shows the cumulative horizontal plastic deformation versus the number of load cycles for the sample which had the strange ER graph (see Figure 4-36). The general shape of the graph can be divided into 3 zones. During the first zone the rate of deformation is nonlinear with the increase of loading cycles, In this relatively short phase the material deforms to dissipate the energy and experiences micro damages; if the amount of stress is lower than the material's threshold, the second phase will be formed in which the rate of deformation became constant and the micro damages start to grow till a point that the material can't dissipate the energy anymore. This phase starts with the sharp increase in the rate of horizontal deformation till the sample break. This graph is robust for FCSMs and can be used as a second complementary method besides the ER graph especially in cases that the ER graph is not clear to define the fatigue point.

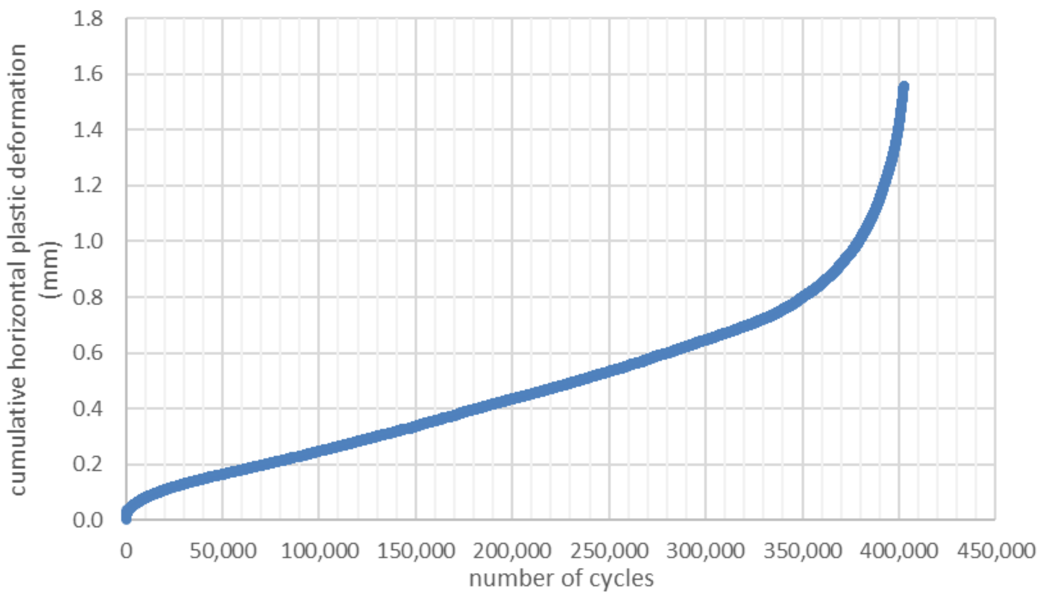


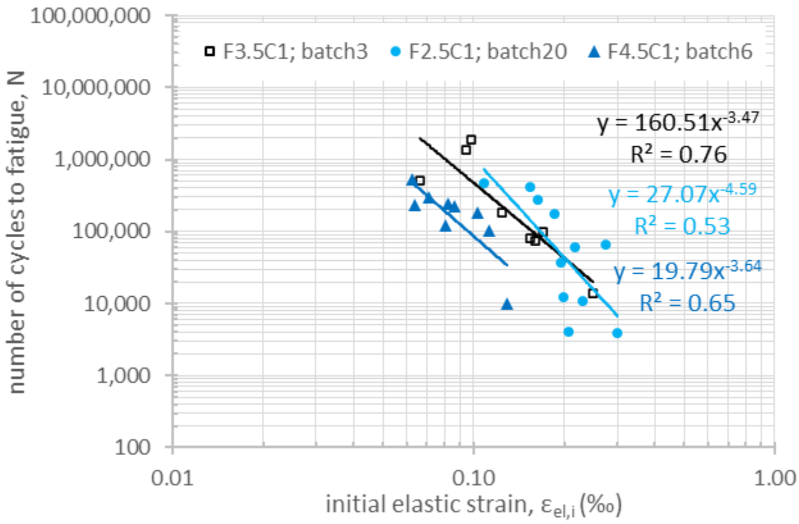
Figure 4-38: Cumulative horizontal plastic deformation vs. number of cycles in fatigue test (sample 3-10 with F3.5C1.0 mix performed at 5 °C, 10 Hz)

If the stiffness amount at the start of the linear part on the graphs of stiffness vs. load cycle is considered as the initial stiffness, the number of cycles that it reaches to 50% of that amount, has a relatively good agreement with the amount determined from ER graph (as the fatigue point). It can be seen by considering Figure 4-35 and Figure 4-37 which are the mentioned graphs for the same sample. The cycles relative to 30% of the initial stiffness amount, have a good agreement with the fatigue life determined from the total horizontal deformation graph vs. loading cycles. These two methods can be used as rules of thumb for an approximation of the fatigue point besides the ER method.

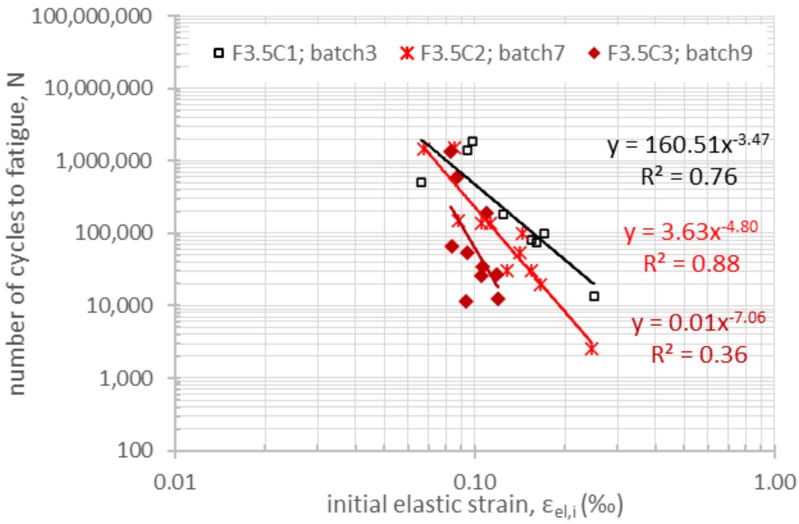
After determining the number of cycles related to the defined fatigue point for each fatigue test set, the acceptable results were used to build the fatigue equation of that set. An equation with the form of $N = C_1 \varepsilon_{el,i}{}^{C_2}$ was fitted to the fatigue life vs. initial strain points. N is the fatigue life of the mix from the indirect tensile cyclic test (the point of macro-damage, known as N_{makro} in Germany), $\varepsilon_{el,i}$ is the initial elastic strain in ‰ which is the average strain amount of the cycles 98 to 102. C_1 and C_2 are the constants that are related to the material's characteristics and are determined by applying the regression equation on the fatigue life vs. initial strain points.

4.11.1 Effect of binding agents' contents on fatigue behavior of the FCSM

To assess the effect of binding agents (bitumen and cement) on the fatigue response, samples with different mix combinations after stiffness-temperatures tests were used for fatigue tests at 5 °C and 10 Hz. Figure 4-39 shows the resulted fatigue curves of different mix combinations. The first thing is that the determination coefficient R^2 of some curves especially the F3.5C3.0 is relatively low compared to the others and the normal amounts for HMA samples. Generally, the fatigue test results of these types of mixes have a higher scatter compared to the HMA and are reported by the other researchers too (see section 2.3.4 in the literature review). Besides that, in cases with higher cement contents, the mix is sensitive to the stress level and therefore it is not easy to cover a wide range of initial strains to get a better determination coefficient R^2. These tests aimed to see how the fatigue behavior changes relative to the amount of binding agents. The effect of cement increase shows a clear trend even with the low amount of R^2 for the F3.5C3.0.



a) Bitumen content effect on fatigue



b) Cement content effect on fatigue

Figure 4-39: Effect of bitumen (a) and cement (b) contents on the fatigue response (tested at 5 °C, 10 Hz)

It is expected that by increasing the amount of bitumen, the mixture becomes more flexible and by increasing the amount of cement, rigidity will increase but it is not always so clear and simple as this. It is the combined effect of both binding agents which determines the resulted mixture's behavior. To be able to consider the combined effect of these two different binding agents, the ratio of bitumen to cement (will be mentioned as B/C) and the ratio of cement to bitumen (will be mentioned as C/B) were used as two indexes. These two indexes change with the changes in the amounts of bitumen and cement. An increase in B/C reflects the shift to more flexibility while an increase in C/B reflects the other direction. For better comparison, the 2 mentioned indexes were calculated for each of the mix combinations and normalized by taking the ones from F3.5C1.0 as the base. The slope of the fatigue line is also a parameter that is affected by changes in the mixture’s characteristics. Therefore it was also normalized based on its amount for the F3.5C1.0 mix. Figure 4-40 shows these 3 normalized parameters for the mix combination. Now it is possible to assess the effect of binding agents on the fatigue of resulting mixes.

By cement increase, the C/B parameter increased for all the cases which is reflected by the increase in the slope of the fatigue line too. In the case of the F2.5C1.0 mix, looking at the amount of normalized C/B explains why the slope of its fatigue line was higher than the F3.5C1.0 mix. In the case of the F4.5C1.0 mix, although the B/C shows that it is flexible than F3.5C1.0 but the slope parameter is a little higher. looking at the stiffness master curves (see Figure 4-20), the F4.5C1.0 mix has relatively higher stiffness than the F3.5C1.0 one and it can be the reason. It seems that the fatigue is not as sensitive to the bitumen content as in HMA mixes. This is mainly because of the different bitumen dispersion and coating in foamed bitumen cold mixes compared to HMA which is mentioned by other researchers too [30].

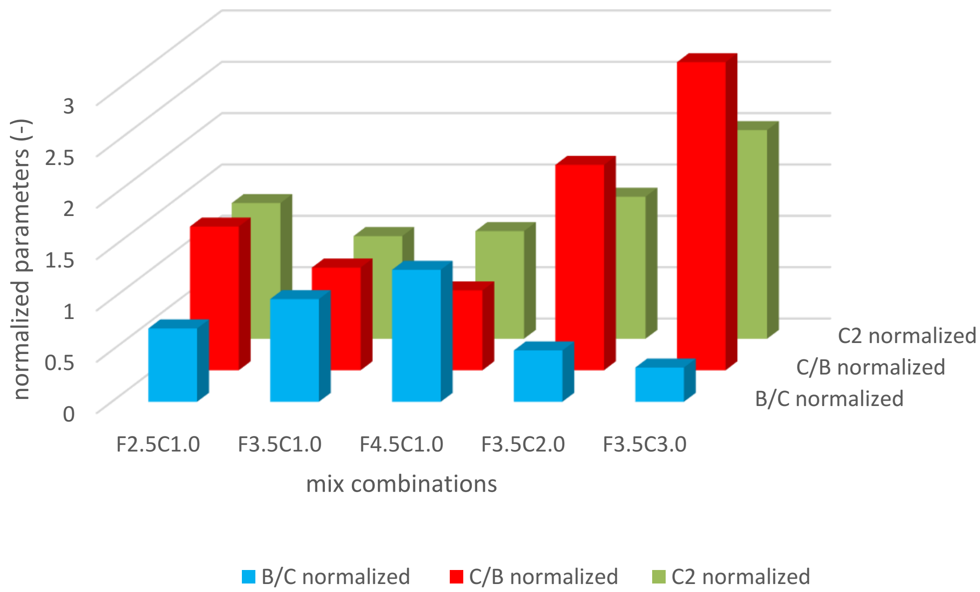


Figure 4-40: The combined effect of mixtures' bitumen and cement contents on the slope of their fatigue lines (B/C: bitumen to cement content, C/B: cement to bitumen content, C2: slope of the fatigue lines), all are normalized based on the parameters of F3.5C1.0 mix

Comparing the fatigue lines of F4.5C1.0 with F3.5C3.0 shows that the addition of cement may improve the fatigue behavior in low strain situations. This observation has been reported by other researchers too (see section 2.3.4 in the literature review).

Looking at the shift of fatigue lines relative to the F3.5C1.0 mix, it seems that higher stiffness leads to a downward shift of the fatigue line. If the higher stiffness is because of higher cement, the shift is with a considerable increase of the line slope too but if the higher stiffness is because of bitumen, the slope change is not too much. This shows that the increase of cement has more effect on the increase of mixes rigidity than the increase of bitumen on its flexibility.

It is important to mention these fatigue lines alone don't mean a mix is better or worse than the other one. They have different stiffnesses at the same strain level; as a simple comparison, in case of equal layer thicknesses, will show different strain responses and therefore different fatigue lives. There are other parameters that affect the situation; as an example, when a thin layer is constructed over a weak foundation, higher stiffness will reduce the fatigue life but if a relatively thick layer is constructed on a strong foundation, an increase of stiffness will usually result to increase of the fatigue life of the mixture.

4.11.2 Effect of curing method

Assessing the effect of fast curing (72 hours at 40 °C instead of standard curing, see section 3.6) on mechanical characteristics of FCSM samples regarding their stiffness and ITS (Figure 4-11 and Figure 4-20), revealed the importance of moisture condition during the curing process. If the oven can't extract the moisture out evenly and properly, high moisture levels combined with relatively high temperature can affect the cement hydration and lead to different results in comparison to the standard curing method (M KRC method). The second point is that the remained amount of moisture in the specimens may be more than the remained amount after standard curing. To investigate if this situation can affect the fatigue response, the F3.5C1.0 samples which were cured with standard and fast methods and used in stiffness-temperature tests were used for fatigue tests too. Figure 4-41 shows the fatigue curves of the mentioned samples with the names of batch 3-S and batch 12-F showing the standard and fast cured samples, respectively. As a control, an extra batch was produced, and the samples were cured with the fast method then tested with the same conditions. Their fatigue curve is also included under the name of batch 18-F.

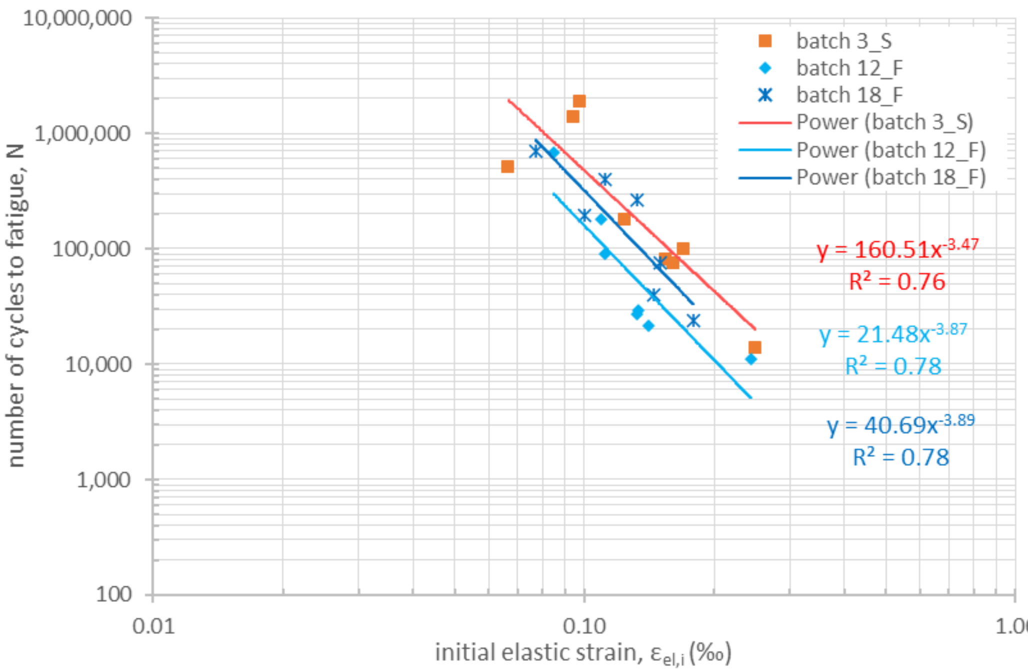


Figure 4-41: The effect of curing method on the fatigue behavior of F3.5C1.0 mixes; fatigue tests at 5 °C, 10 Hz (S: standard cured based on M KRC and F: fast cured, 72 hours in 40 °C)

Comparing the graphs together shows that the curing method may affect the fatigue response too. Fatigue lines of fast cured batches (12 and 18) are shifted downward and have a little higher slope compared to the standard cured batch (3). Looking to the stiffness master curves (see Figure 4-20) shows higher stiffness for fast cured specimens (batch 12) compared to the standard cured ones (batch 3). Putting these two observations together confirms the previous conclusion that the higher stiffness shifts the fatigue line downward. The increase of the fatigue line slope confirms the already hypothesis that the applied fast curing has led to a little better hydration of cement in these samples. The second point is a good agreement between the slopes of the fatigue lines from batches 12 and 18, which shows relatively good repeatability of the production and the test.

The average cured density of the batch 3 samples was 2.36 gr/cm^3 and for batches 12 and 18, were 2.34 and 2.28 gr/cm^3, respectively. As they were cured under two different methods with different durations and the density was determined after curing, maybe the amount of remaining moisture was different between them and therefore it is not possible to have any interpretation of density effect on fatigue results.

It is important to mention again that these observations don't mean that the fast curing always leads to the higher stiffness or a shifted fatigue line, they just highlight that the curing method's parameters (i.e., relative moisture conditions, temperature amount and duration) can affect the resulted material's characteristics.

4.11.3 Effect of test temperature

Test temperature is a parameter that affects the material's response to load and therefore the fatigue life of the mixes. Understanding this effect is important for defining appropriate shift factors to link laboratory life to field life. To assess the possible effect of temperature, two extra batches with F3.5C1.0 were produced and cured with the standard method (batches 13 and 14). These samples were tested at temperatures 0, 10 and 20 °C.

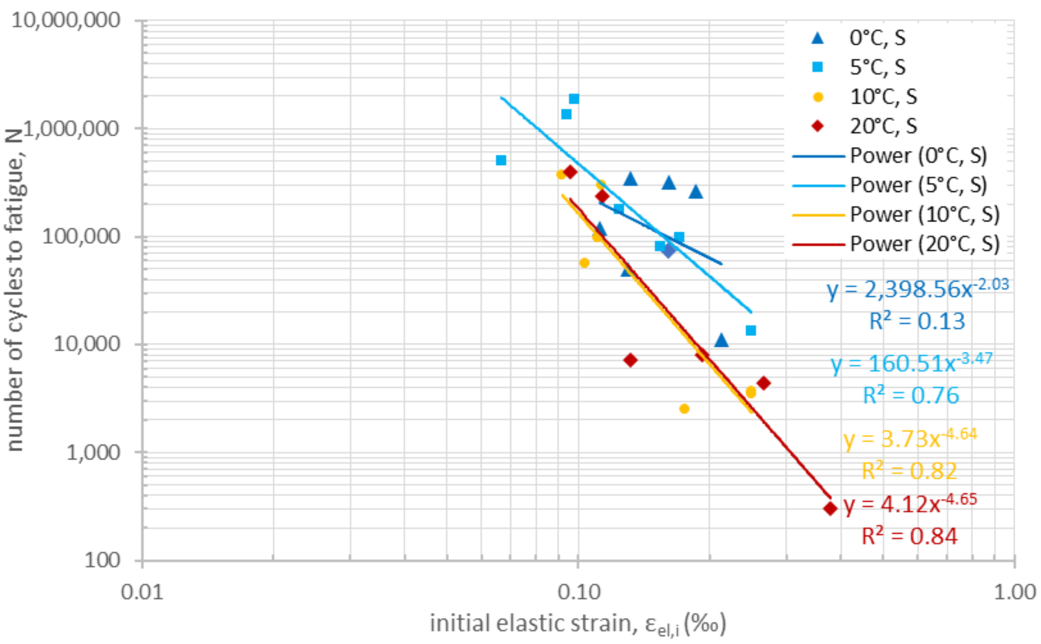


Figure 4-42: Fatigue curves of F3.5C1.0 mix samples tested at different temperatures and 10 Hz frequency (S, stands for standard cured)

Figure 4-42 shows the resulted fatigue lines plus the one from the already made tests at 5 °C on the samples with the same mix combination and curing method. Looking at them, the first point of interest is the high scatter at 0 °C. Looking at the broken surface of the samples didn't show any remarkable difference between them. As mentioned before, the remained moisture in the specimens never reaches zero and is not homogeny distributed in the specimen. In low temperatures, it freezes and can affect the results. Looking at the scatter of the result points, they can be divided into 3 points as the upper and the other 3 as the lower lines. This makes it impossible to decide on outliers therefore the zero temperature needs more test results. Considering the other 3 temperatures, it seems that the test temperature affects the results. 10 and 20 °C results are almost the same but compared to 5 °C, a shift is clear with a decrease of the line's slope. This is due to the fact that the material behaves less visco-elastic at lower temperatures and more elastically. By decreasing the test temperature fatigue life increases at the higher strains but decreases at the lower strains.

Figure 4-43 shows the fatigue lines for F2.5C1.0 mixes at two different temperatures of 5 and 20 °C. The same trend as in F3.5C1.0 mixes can be seen for this mix combination too. Lower test temperature shifted the fatigue line the same as the F3.5C1.0 mixes (Figure 4-42) and decreased the slope. Regarding the lower R^2 amount of the line at 5 °C, outlying two points may lead to a higher R^2 (0.77) and change in line parameters (C_1 = 7.67 and C_2 = -5.37) but important is that with the resulted fatigue line, still the same shift trend and decrease in slope is valid. As an extra control, one test was done at 10 °C. The position of the resulting point regarding the 20 °C line, shows the same observation as in the F3.5C1.0 mix.

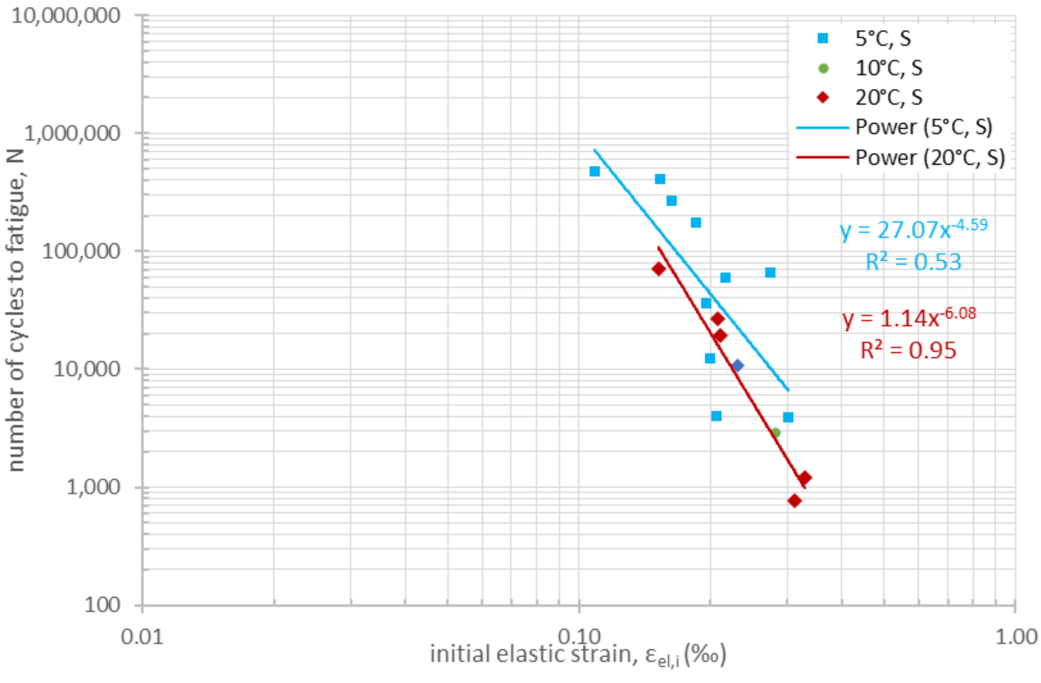


Figure 4-43: Fatigue curves of F2.5C1.0 mix samples tested at different temperatures and 10 Hz frequency (S, stands for standard cured)

The same response and behavior were also reported in the case of indirect tensile fatigue tests on HMA samples when instead of total deformation, resilient (recoverable) deformation is taken to determine the initial strain [197, 198, 199]. To compare the results of this test type with flexural tests (like 4-point beam fatigue), especially regarding the effect of temperature or stiffness of the mixes, total strain should be used instead of resilient strain. These test results show that the fatigue life is affected by the temperature but integrating that in the pavement structural design procedure requires more fatigue and low-temperature behavioral tests.

4.11.4 Assessment of the indirect tensile fatigue test method for FCSM mixes

The first step in assessing the appropriateness of the indirect tensile fatigue test (ITFT) method for FCSM mixes is to consider the parameters which are the sources of the test's results variability. It is possible to group them under three main categories:

1- Related to the material's inherent characteristics,
2- Related to the specimen's production and preparation procedures,
3- Related to the testing method and its specifications

FCSMs are multi-phase hybrid materials with non-continuous bonds which show a combined behavior of elasto-plastic and visco-elastic materials affected by different parameters mainly their binding agents, parent material's characteristics and their curing history. This level of complexity especially the different types of bonds and their non-continuous distribution are the main cause of heterogeneity.

The main influencing factors in mixture production are changes in the parent material, especially the finer than 2 mm fraction, material's mix temperature, consistency of foam bitumen production, mixing instruction, moisture contents during mixing and compaction. In the specimen production and preparation part, the size and shape of the samples, operator's experience, curing method (consistency of moisture and temperature history) plus the temperature conditioning of the samples are the affecting parameters. By a regular and systematic control of the equipment and parent materials and by applying a fixed instruction for production and preparation, it is possible to minimize the effect of these factors. Each research reveals some new important factors and it is important to properly document them to be able to consider them in future researches.

In the case of the testing method, diametrical loading of cylindrical specimen results in a complex stress distribution state in the sample. Interaction of this stress state with the heterogenic nature of the FCSM affects not only the failure initiation point but also its type and growth model in the sample. A variety of scenarios with different starting damage types, initiation points and growing patterns are possible to occur. In low cement contents (1% and less), the free filler phase will be in some parts lightly cemented as the cement is not enough to form a separate bond phase. Under loading, it is possible that these weak bonds damage not only in tension but also in compression too. This will cause the aggregate reorientation and the formation of a new microstructure with an overall decrease in stiffness. Depending on the load magnitude, the next step will be the shear failure in the aggregate phase (and also in the damaged filler phase) in that area of the sample. The outer area of the sample may act as an arc over the inner area and becomes resilient under load repetitions when the load magnitude is not big. In this case, a long fatigue life will show by the test results, but it is not classic pure fatigue damage. The second possibility may occur when the loading magnitude is high enough to break the arc and succeeds to initiate tensile fatigue damage, but it will stop growing and extending (because of the noncontinuous nature of the material) therefore, there won't be a main crack but different damaged areas which may be connected to form a macro-damage and later failure by the test progress. It is also possible that in some cases, the distribution of the bonds is in a way that the initiated crack can grow enough and its growth can't be postponed. In these cases, a more classical fatigue will occur. If the load magnitude is high enough to initiate the tensile crack in the middle zone of the sample and simultaneously a compressive failure with enough depth, the arcing phenomenon won't occur and will result in shorter fatigue life. The mentioned examples are some simplified and separate possibilities, but the real state is more complicated and mixed with different damage types. In higher cement percentages (2 and 3%), more bonded phases will form but depending on the strength ratio of bituminous and cement-bonded phases, the damage initiation and its growth from micro-damage to macro-damage will differ.

The above short explanation shows how the interaction of the material's heterogeneity and the test type can lead to the variation of the results which has been reported by other researchers too (see section 2.3.4 in the literature review).

To evaluate the variability of the fatigue results, a batch with F3.5C1.0 mix was produced, cured with fast curing method and subjected under the same initial stress in the fatigue test. The tests were performed at 5 °C, 10 Hz. The first 3 samples were used to find a stress amount which leads to initial elastic strains around 0.1‰. Table 4-18 shows the results.

Table 4-18: Fatigue tests results of the batch 15 (F3.5C1.0) at 5 °C, 10 Hz

Samples ID	Upper stress (MPa)	$\varepsilon_{el,i}$ (‰)	Stiffness (MPa)	Fatigue life
15-1	0.40	0.249	2,353	5,508
15-2	0.38	0.160	3,513	17,701
15-3	0.34	0.125	3,941	54,800
15-4	0.32	0.060	7,634	71,308
15-5		0.092	5,070	85,000
15-6		0.096	4,673	193,510
15-7		0.105	4,341	236,004
15-8		0.139	3,262	73,210
15-9		0.108	4,303	52,906
15-10		0.100	4,604	58,204
Coefficient of variations, CV (%)		14%	13%	61%

By looking at the results, it is clear that the 15-4 result is an outlier because the fatigue life doesn't match the stiffness and the initial strain amount so, it wasn't considered in the calculation of statistical parameters. Also, as the first 3 samples were tested at different stress levels, they weren't used too. The high amount of CV for the fatigue life of the samples shows how much the results of the fatigue test can vary even at the same initial stress. Another point is the lower amount of CV for the resulted initial strain and stiffness. This observation may show that it is better to use more than 9 samples for a series of fatigue tests on these mixes. Twagira [30] used 12 samples for 4-beam fatigue tests and mentioned that will result in a good level of significance.

Figure 4-44 shows the fatigue line of the batch 15 specimens (into which 3 extra fatigue test results at other stress levels were also included) in comparison with the two other fast cured F3.5C1.0 batches (12 & 18 which were produced to assess the effect of fast curing method;

see section 4.11.2 and Table 3-1). The aim is to assess the variation of the results when material, operator, production and testing machines are the same and the main difference is the material's heterogeneity. Looking at the fatigue lines and comparing them together shows a sufficient level of agreement. The difference can be possibly because of different moisture content history during the fast curing. These results show that the fatigue tests had a useable level of reproducibility in one laboratory.

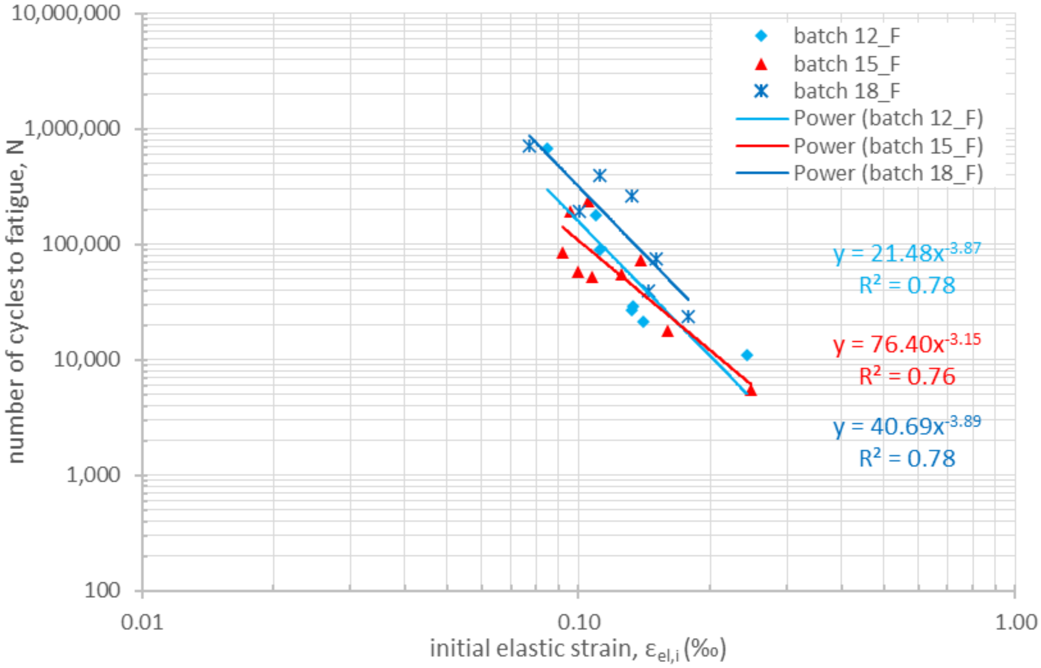


Figure 4-44: Fatigue line of the 3 different batches of F3.5C1.0 mixes; tested at 5 °C, 10 Hz (F, stands for fast cured, 72 hours in 40 °C)

To assess the effect of more variables on reproducibility of the results, a batch was produced in the German Federal Highway Research Institute (BASt) after almost 2 years with the same aggregates, filler, cement and bitumen transferred from Siegen. The aggregate fractions and filler were mixed with adjusted percentages to be sure the resulted gradation is the same as before. They were used to produce the F3.5C1.0 mix combination (batch 24). Foamed bitumen and mixture were produced with the same parameters and settings (i.e., same temperature, foaming water amount as well as the same mixing instructions) but on another set of foaming and mixing equipment (a newly bought set of Wirtgen WLB10S foaming machines and WLM30 twin-shaft mixer by BASt). Samples were compacted with the available Marshall compactor (with the same compaction procedure) and cured with the same standard method (M KRC method). The first two days after demolding, they were put in a temperature cabin that had the possibility of automatic regulation of relative humidity. Tests were performed with BASt's dynamic testing machine (Dynam-IT-M, Nr. 03005, the old name: Josef Freundel). After curing, the samples first preloaded at 20 °C and 10 Hz in steps to strain levels up to 0.08‰, then used for stiffness tests (the results are already depicted in Figure 4-30) and at the end for fatigue tests at 20 °C, 10 Hz. The fatigue life was determined based on TP Asphalt-StB (Teil 24), with the ADtoPave® software. It is possible to say that all types of variability sources were tried to be included in these samples. These samples will be named as BASt samples compared to the others as Uni.-Siegen samples.

The average density of the samples was 2.34 gr/cm³, the same as the average of the Siegen samples for the 20 °C, fatigue tests.

Figure 4-45 shows the results of the fatigue tests of BASt samples with circle markers (black and red colors). For comparison, the similar results (20 °C, 10 Hz) from Uni.-Siegen (triangle markers) plus all results of the F3.5C1.0 mix samples from Uni.-Siegen (different curing methods and test temperatures) are also added to get an insight on the range of them.

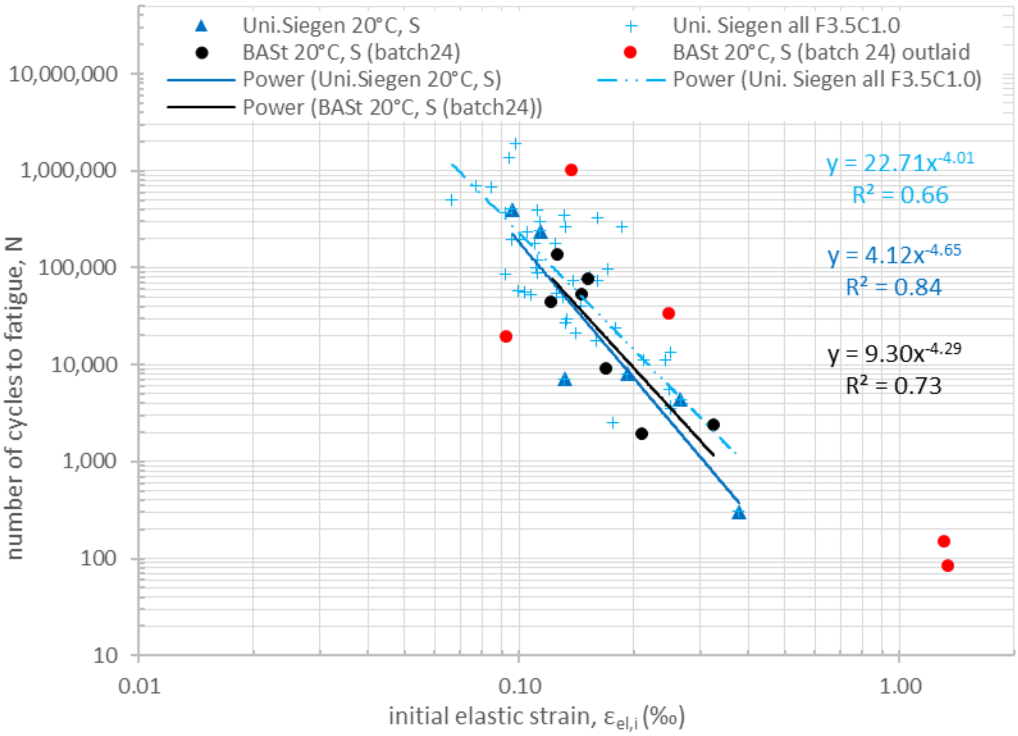


Figure 4-45: Fatigue lines of F3.5C1.0 mixes made and tested in 2 different laboratories

By the first look, it seems that the BASt results are very scattered, but except for the two points which had a very big initial strain of more than 1‰ and shouldn't be considered, the rest are in the range of the F3.5C1.0 results. This shows the material's inherent heterogeneity can even mask the effects of the test's temperature or specimen curing history. A second and more detailed look at the results shows that some points can be outliers. By exclusion of them, the fatigue line was driven (the black line). Comparing the BASt fatigue line and its relative from Uni.-Siegen shows a good level of test reproducibility between the two laboratories. This example revealed an important point in interpreting the results and extracting the fatigue line. It shows how important is to correctly decide on the outliers.

It seems that pre-loading and performing cyclic stiffness tests on the samples with relatively high strain levels (which are normal for HMA), didn't affect the fatigue results. As all the fatigue tests were done with higher initial strains than the ones for stiffness tests, therefore they weren't affected by pre-loading or stiffness tests. This is also in agreement with the behavior observed in multi-step stiffness tests. It is important to mention that these are only one set of tests and for a concrete conclusion it is better to perform more test rounds.

4.11.5 Comparing the fatigue behavior of foam mixes with HMA

To have a comparison between the hot mix and cold mixes with foamed bitumen, the reference hot made samples with the same mix design of F3.5C1.0, which were tested for stiffness at different temperatures and frequencies (see Figure 4-20), tested in fatigue at 5 °C and 10 Hz. Figure 4-46, shows the resulted fatigue line (named HM-3.5-1) plus the fatigue lines of the F3.5C1.0 mixes at the same temperature but with different curing methods.

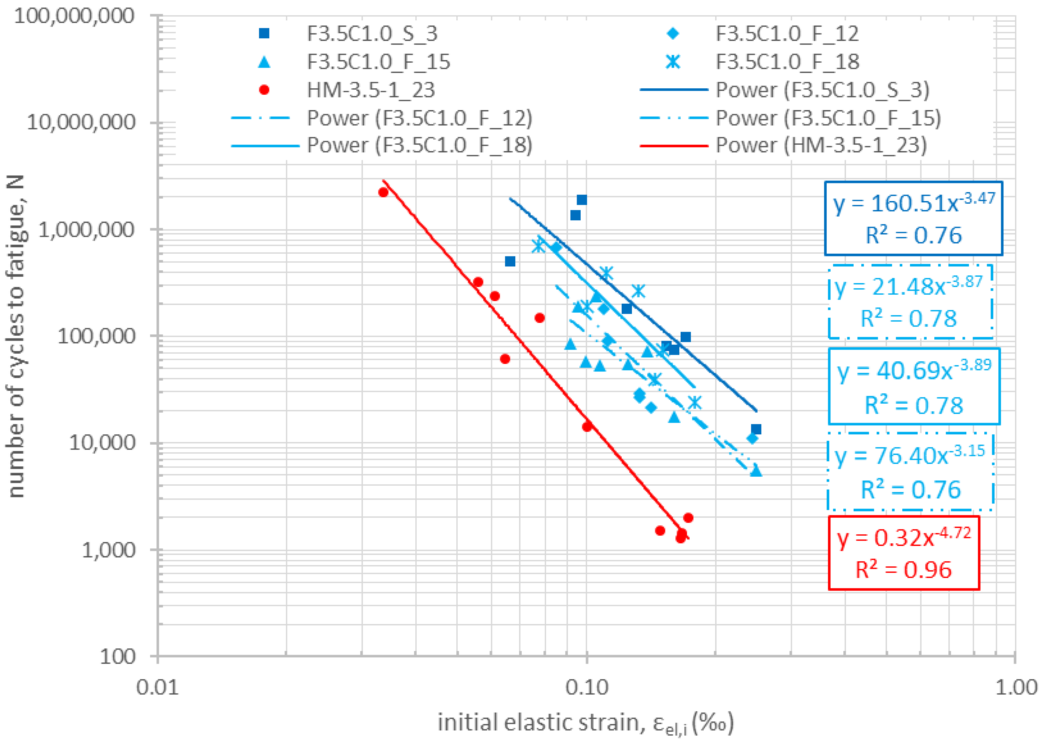


Figure 4-46: Fatigue lines of the hot made mixture (HM-3.5-1) in comparison with F3.5C1.0 cold mixes tested at 5 °C, 10 Hz (S, stands for standard cured and F, for fast cured; the last number shows their batch). Note that the hot made specimens have higher stiffness at the test temperature (see Figure 4-20)

Considering that the hot made specimens were the same as the cold ones (same aggregate grading, mix combination, specimen size and compaction method), the higher R^2 amount for the fatigue line of the hot made specimens shows that the selected sample's size (Standard Marshall sample size with 101 mm diameter for the maximum aggregate size of 22.4 mm), is not a serious determining factor in the variation of the tests' results for FCSMs. On the other hand, the big difference between the F3.5C1.0 and HM-3.5-1 is not in the aggregate phase pack but the bituminous mastic phase characteristics. With the same amount of bitumen, as it covers the surface of all aggregates and filler in the hot mix compared to the selective coating in the cold one, then the thickness of bitumen coating is lower. On the other hand, the non-continuous bituminous bonds (or the bituminous mastic) are richer in bitumen and therefore they respond differently to the fatigue.

Comparing the position of the HM-3.5-1 fatigue line with the F3.5C1.0 ones shows a shift down and little increase in the slope of the line. Looking at the stiffness master curves (see Figure 4-20), the stiffness of the hot made samples are considerably higher than the foamed bitumen mixes at 5 °C. This agrees with the already made conclusion as when the stiffness of the mix (at the fatigue testing temperature) is higher than the other one, then its fatigue line will shift down with an increase in the slope but as the higher stiffness is not because of higher rigidity, therefore the slope increase is not big. The same type of relation between stiffness and the fatigue line parameters (resulting from indirect tensile test mode) has been reported for HMA by Dragon and Wellner [200].

It is important to mention again that these results shouldn't be interpreted as a ranking tool to compare the fatigue performance of these two types of mixtures together. When a mixture has higher stiffness (hot made samples), higher stress should be applied to the sample during the fatigue tests to get the same strain level as the samples of the mixture with lower stiffness (foamed bitumen cold mixes). If instead of initial strain, initial stress is used to draw the fatigue line, then the results will show that the hot mix has a better fatigue performance.

4.12 Damage mechanism of the FCSM

Considering the observations during the tests and the results of the assessments, this section tries to summarize them together and to introduce a damage mechanism for the foam bitumen-cement stabilized / recycled mixes.

The first important point and finding is that when foamed bitumen is selected and used as the bituminous binder with cold mixes production method, the bituminous bonds in the resulted material will always be non-continuous but based on the cement's amount, two scenarios may occur.

First, at low cement contents

When the amount of cement is under the limit to form a bond microstructure (in the ranges of ≤ 1%), the material will have a weak (and non-continuous) bond state in the filler phase too. Under the action of stresses (below the indirect tensile strength of the material), these bonds will damage preliminary with a fatigue failure type but because of the non-continuous state of the material, the damage can't grow and propagate with the mechanism of propagation in continuum spaces (with the Paris law). It will be like isolated damaged zones in the material which result in to decrease in stiffness and plastic deformation. This can be seen in the first part of stiffness or cumulative horizontal permanent deformation versus loading cycles graphs during the fatigue tests (see Figure 4-37 and Figure 4-38).

If the amount of stress is lower than the limit which can be dissipated through this material's reaction, a kind of shakedown will occur in the material and a new microstructure will form with a resilient response. This could be seen by looking at the force versus horizontal deformation of the indirect tensile tests (before and after the cyclic multi-step stiffness tests, section 4.8.1). After that, the rate of damage will become lower (compared to the first stage) and relatively constant afterward. The material will show a resilient behavior till these cumulative damages reach the total capacity of the material. This state can be seen with the low rate of stiffness decrease versus load cycles or low rate of cumulative total horizontal strain growth with load cycles (see Figure 4-37 and Figure 4-38). The formation of this state and its duration depends on the amount of imposed stress and the capacity of the material to dissipate it.

In mixes with relatively high bitumen content (≥ 3%) the damage will then be a combination of fatigue (primarily) and plastic deformation (secondary). In mixes with lower bitumen amounts (in the range of BSM definition), the type of damage is relatively the same; but because of the higher effect of bituminous bond's non-continuity (resulting from a lower amount of bitumen), plastic deformation (permanent deformation) is the dominant in-field observable damage mechanism.

It is possible to conclude that in foamed stabilized mixes, classical fatigue as is known from continuum spaces theory (which is assumable for HMA or concrete) will not occur. Considering this two side behavior of these mixes, it is possible to produce mixes with a good performance against cracking and permanent deformation by balancing the contents of the binding agents (bitumen and cement) based on the aggregate pack skeleton and simultaneously with optimizing the pack by considering the gradation and shear strength characteristics of the parent material.

Second, at high cement contents

When the cement content is enough to form a bond filler phase, the material will change to a relatively continuous state (can be assumed as a continuum) and therefore, the classical fatigue and damage growth will dominate the failure mechanism. It seems that 3% cement can lead to the formation of this state. Cement contents between 1.5 to 3% will result in different responses which are not always the same and not easy to interpret because of the interaction of many parameters.

Chapter 5

5 Structural Design of Pavements with FCSM Base Layers

This chapter aims to deal with the last question of this research (see section 3.1) by addressing the main issues when the analytical approach is applied for the structural design of pavements having FCSM layers. The chapter will start with the main steps of the analytical approach for pavement design mainly based on the existing approach in Germany for flexible pavements, known as RDO Asphalt 09 [201]. The issues related to the FCSMs will be explained in each step by considering the findings from the tests and assessments and recommendations will be given to deal with them. Stiffness behavior and how to select the layer stiffness has been discussed as the main issue for analytical design.

Shift factor as the second main topic has been considered with a simple approach to select an amount for it. The procedures for determining the input parameters and the shift factor were also utilized for the F3.5C1.0 mix. Because of the low amount of cement (equal to 1%), the mix is not rigid and hydraulically dominant. The bitumen content is also high enough (more than 3%) that the fatigue failure type can be considered as the material's preliminary failure mode. Results of different stiffness and fatigue tests confirm this selection. Considering the literature review, this mix combination is very common when virgin aggregates are the main parent material (100% virgin aggregates or mixes with up to 20% RAP) and a sort of bound behavior is expected from the mix. The approach was used later to design pavement sections for 7 loading classes of RStO 12 [202] with the F3.5C1.0 material.

At the end of the chapter, based on the applied method and the gained experiences from tests, a step-by-step design procedure is recommended with the number of needed specimens and the tests types.

5.1 Steps of the analytical approach based on RDO Asphalt 09

As the main approach of this research is to check the possibility of applying existing methods in Germany, the analytical method for the flexible pavements known as RDO Asphalt 09 [201] is selected as the base for the structural design. Considering the international approaches for pavement design, this method is a mechanistic-empirical (ME) approach and has three main steps.

5.1.1 Input parameters

Traffic data, climate data and conditions, material characteristics and pavement parameters are the main input parameters. The main pavement parameters are the layer's thicknesses, type of them, their position and their interface bond state. Material parameters are mainly the mechanical characteristics of the material which depending on the pavement's model and its complexity, can be more to consider more detailed behaviors. In most pavement design methods, stiffness and Poisson's ratio are the main inputs to model the material. The second group of the material parameters are performance related which are generally different types of transfer functions that transfer the responses of the material in the pavement to its life. Depending on the failure type (or types) of the material, different types of transfer functions are defined mainly from the results of laboratory tests.

Multi-layer linear elastic is the most common and practical method of modeling and analyzing the flexible pavement layers and has been recommended by RDO Asphalt 09 too. For hot mix asphalt material, the stiffness is temperature and loading rate dependent and to consider that, normally stiffness master curve is applied to determine the stiffness at desired temperatures (and 10 Hz as the normally used frequency). In the case of granular material (granular base layers and frost protection layer) and the subgrade, the elastic modulus is used. The layer modulus in combination with its thickness and the underneath layer, should satisfy a minimum required modulus of reaction which is controlled with a static plate load test on top of the layer (known as E_{v2}). In FCSM mixes, the stiffness is time and temperature dependent and also is affected by the level and history of horizontal strain (or the load which causes the strain). How to deal with this behavior and how to bring it into the pavement design process, is explained in the next section.

In Practical cases, Poisson's ratio is taken normally constant (0.35 for hot mix asphaltic layers and 0.5 for granular layers and subgrade). For the FCSM mixes in the ranges of this research, it is recommended to use the constant amount of 0.28 to 0.3 as the input for the Poisson's ratio of the layer.

5.1.2 Pavement model and analysis

This step aims to determine the response of the pavement material to the external actions resulting from loading or climate. The pavement model will be built and analyzed to determine the responses at critical points. Depending on the type of material and the aims of modeling, different material models and different analyzing types can be utilized. One of the most employed ones for practical purposes is the multi-layer linear elastic model, mentioned before. Considering the temperature dependency behavior of bituminous mixes, a simple way to take it into consideration in linear elastic models is to take different temperature analysis cases and determine the stiffnesses for each temperature case. As the temperature will change by the depth and is not equal to the surface temperature of the pavement, based on comprehensive studies on the change of temperature in the depth of the flexible pavements at different times of the year in Germany, 13 different surface temperatures classes with their statistical distribution over the year have been provided in RDO Asphalt 09 [201] for 4 different temperatures zones in Germany. There is also an equation that determines the temperature amount in any depth of the pavement when the surface temperature is known. The bituminous layers can be divided into thinner layers (sub-layers) and the temperature in the middle of that layer to be determined and used for calculation of the stiffness of that layer. Sub layers of 1 to 2 cm thickness are recommended for HMA as the stiffness is more sensitive to temperature changes. This approach can be utilized to foamed bitumen stabilized mixes too if the analysis program has the possibility of that. If this is not provided, considering the temperature in the middle of the layer and the horizontal strain at the bottom will lead to conservative results and is acceptable. If the traffic is in the form of different axle classes, then for each temperature and axle class the model should be analyzed (as an example, based on RDO Asphalt 09, 11 axle classes and 13 temperature cases may be considered). When there is no traffic data available, the design traffic can be considered in 10-ton equivalent single axle load (with a circular contact area of 15 cm radius).

5.1.3 Controlling the pavement capacity

In this step, the capacity of each layer (regarding its material) is determined and controlled with the needed (required) capacity. The existing capacity will be determined for each analysis case by considering the critical responses and the help of appropriate transfer functions. It is adjusted to in-filed expected capacity with shift and safety factors. The critical responses are selected based on the type of the material and its failure mode / modes. On the other side is the required capacity which is the number of load repetitions in that analysis case. The ratio of the required capacity to the existing capacity is determined for each combination of load and temperature case and summed up to determine the overall damage index of that response (Miner law). If the index is ≤ 1 (or ≤ 100%) then the design is approved for that layer's failure type. If not, the pavement section should be reviewed again and after appropriate justifications (material or geometry), the steps should be repeated.

When foamed bitumen mixes are contributed to a pavement layer, as mentioned before (see section 4.12), depending on the amount of the binding agents, either fatigue or permanent deformation can be the primary failure types accordingly. For the F3.5C1.0 mix combination, fatigue can be considered as the primary failure mode and the tensile strain at the bottom of the layer is the critical response to determine the capacity (life) for this failure type. By having the fatigue function determined from the laboratory fatigue tests, it is possible to transfer the response to life; the missing part is the shift factor for this material type which is addressed later in a separate section in this chapter.

5.2 Stiffness model of FCSM

Considering the results of different types of stiffness tests, the FCSM mixes have temperature (and loading rate) dependency (see stiffness master curves, section 4.6) and also horizontal strain level dependency (multi-step stiffness tests, section 4.8.1). Different approaches can be taken for selecting the design stiffness, from simple constant stiffness to a temperature and strain level dependent model. If the stiffness tests are decided to be performed at horizontal strain rages for HMA (0.05 to 0.1‰), firstly all the specimens should be conditioned (loaded till the desired horizontal strain range at 20 °C and 10 Hz) in order to be sure that they all have the same state of inter-damage before testing them at different temperatures and frequencies. This is a relatively complicated process that is not practical in real daily applications and is not recommended. The second point is that applying higher strain levels has the risk of affecting the fatigue test's results when the same specimens are used for the fatigue tests too.

As mentioned already, during this research, the stiffness tests for construction of the stiffness master curves, were performed in relatively low horizontal strain ranges to be sure that the specimens are not damaged and are usable for the fatigue tests too. This point is normal and should be considered during the stiffness tests on FCSMs when the tests are meant to be used for determining the material's stiffness master curve and the specimens are used later for the fatigue tests. On the other side, if this stiffness master curve is used as the input for structural design purposes, the effect of the test's lower horizontal strain level on the resulting master curve, needs to be corrected. To do that, the stiffness-temperature model needs to be justified for other strain levels which are desired for the pavement designer.

Based on the literature and analysis results of the specimens' stiffness at different horizontal strain levels (up to 0.14‰) during this research, the micro damages seem to initiate first in the free / weakly cemented filler phase. Therefore, it is acceptable to assume the distribution of the bitumen doesn't change in this stage. As the amount of that also doesn't change, therefore, it is acceptable to assume, the temperature and strain dependency of the stiffness are independent parameters. Looking at the graphs of stiffness dependency to temperature for F3.5C1.0 mix combination for the two states of low and high range horizontal strains (see Figure 4-30), the temperature dependency of the material didn't change too much. It can be explained with the above-mentioned points and confirms that hypothesis. Considering this assumption, it is possible to define the stiffness as the sum of two independent functions of temperature and horizontal strain with the below general equation:

$$S = f_1(T) + f_2(\mathcal{E}) \qquad \textbf{Equation 5-1}$$

S, stiffness of the material in temperature T and horizontal strain level of ε (N/mm^2)

T, temperature of the material (°C)

ε, horizontal tensile strain level (‰)

By having the stiffness-temperature relation from the stiffness master curve (the tests at low to middle ranges of horizontal tensile strain) and the results of a series of multi-step stiffness tests (only at one temperature and frequency), it is possible to combine them and to determine the general equation of the stiffness. The procedure can be done in 4 steps.

Step one- defining the stiffness-temperature equation

From the stiffness master curve, the stiffness can be calculated at temperatures ranging from -20 °C to 50 °C (-20, -10, 0, 5, 10, 15, 20, 35, 50) and frequency of 10 Hz. A polynomic type of equation with the order 5 (see Equation 5-3) is then fitted onto the data and its constants will be determined with 7 decimal places. This equation will be the first term of the general stiffness model (the $f_1(T)$ in Equation 5-1). Figure 5-1 shows the polynomic equation parameters that were calculated for the F3.5C1.0 mix combination from its stiffness master curve (with the asphaltic Poisson's ratio model).

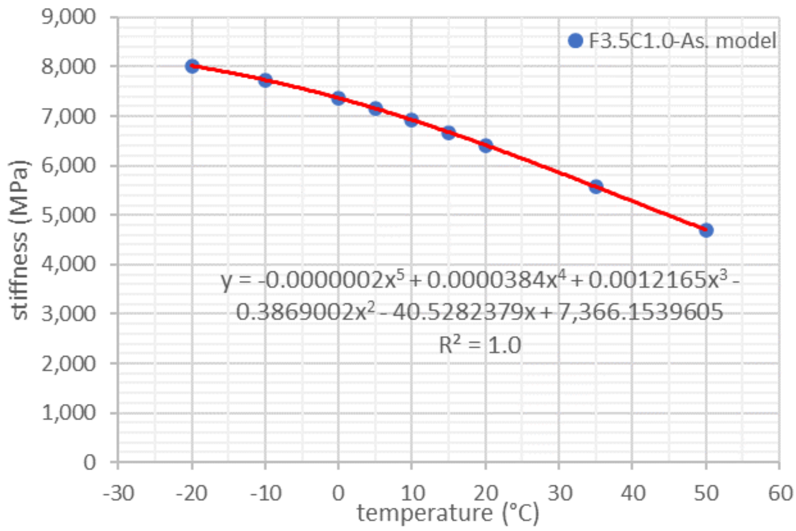


Figure 5-1: Stiffness-temperature equation of the F3.4C1.0 determined from its stiffness master curve in polynomic form (the As. model shows the Poisson's ratio model of the master curve)

Step two- defining the stiffness-strain level equation

In this step, by using the results of multi-step stiffness tests on a separate set of specimens, the relation of the stiffness with strain level is determined. As shown before in section 4.8.1, the trend can be defined with a power type equation stiffness = a*(strain level)^b. It was also shown that the parameters of the equation can be determined at different levels of confidence instead of using the average of the tests' results on the specimens. t-student distribution with 90% (one-tailed) is recommended for the calculations with 4 decimal places for the equation parameters. The tests can be done at any temperature but with the same loading frequency as the step one equation. The stiffness should be calculated with the same Poisson's ratio model (amount) which was used in step one. Figure 5-2 shows the multi-step stiffness tests on the F3.5C1.0 (batch 21) and the stiffness-strain level equation determined from them.

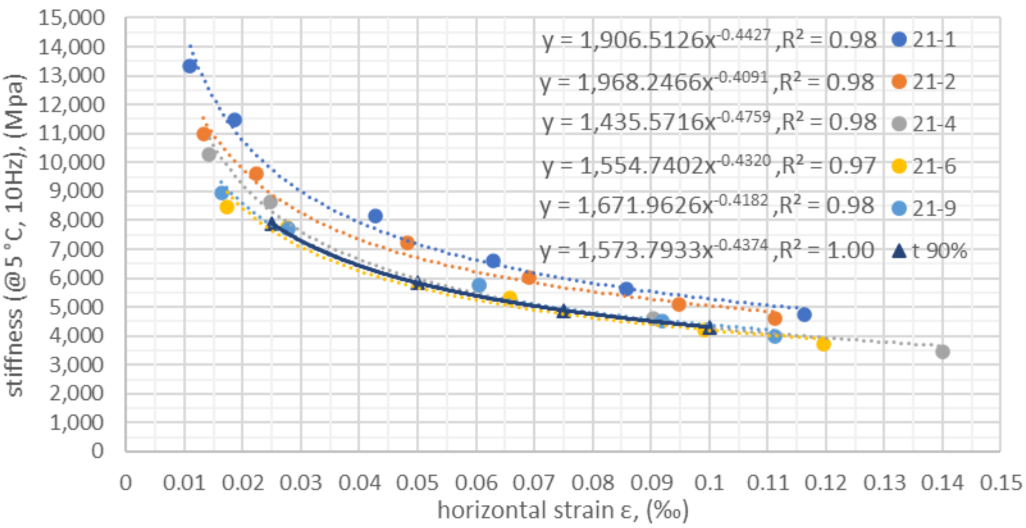


Figure 5-2: Multi-step stiffness tests of F3.5C1.0 samples (at 5 °C and 10 Hz) and the stiffness-strain level equation determined from them by t-student at 90% level of confidence

Step three- defining the second term of the general equation

In this step, the step two equation is used to calculate the stiffness at different strain levels (named as ε_i) and the step one equation is used to determine the stiffness at the temperature which the step two tests were done in that. Then, the difference between these two stiffness amounts (named as ΔS), is determined for each of the strain levels (ε_i). The relation between the ΔS and the ε_i can be defined as a function with the below form:

$$\Delta S = A\mathcal{E}^{B} + C\mathcal{E} + D \qquad \textbf{Equation 5-2}$$

A, B, C and D are the constants that can be determined through a regression procedure by minimizing the sum of normalized errors. Figure 5-3 shows the stiffness difference amounts from the calculation and the fitted equation for the F3.5C1.0 mix (calculated constants are in Table 5-1).

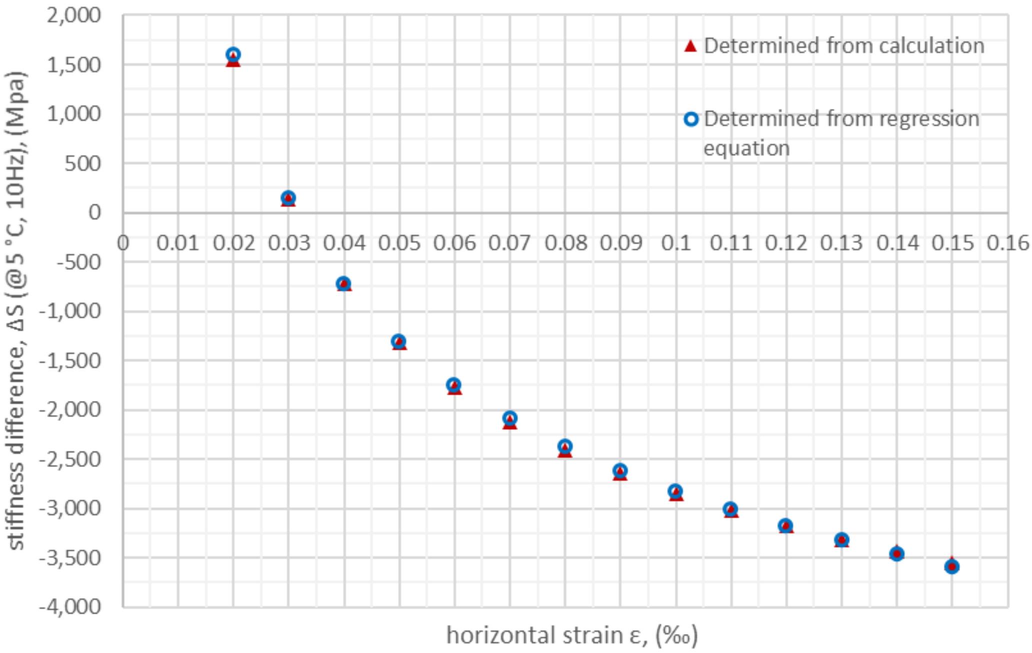


Figure 5-3: Stiffness difference (ΔS) at different strain levels, from calculation and regression

Step four- defining the general equation

Now the terms of the general equation (Equation 5-1) can be determined as:

$$f_1(T) = a_1T^5 + a_2T^4 + a_3T^3 + a_4T^2 + a_5T + a_6 \qquad \textbf{Equation 5-3}$$

$$f_2(\mathcal{E}) = A\mathcal{E}^{B} + C\mathcal{E} + D \qquad \textbf{Equation 5-4}$$

The a1 to a6 are taken from step one and the A to D are taken from step three.

The above-mentioned steps were applied to the tests' results of the F3.5C1.0 mix combination. For determining the stiffness-temperature equation, the stiffness master curve of the standard cured samples with the asphaltic model Poisson's ratio was used. For the stiffness-strain level equation, the data from multi-step stiffness tests of batch 21 samples performed at 5 °C and 10 Hz, were used. Table 5-1, shows the equations' parameters of the stiffness model.

Table 5-1: Parameters of the stiffness model calculated for F3.5C1.0 mix combination

$f_1(T)$ parameters		$f_2(\mathcal{E})$ parameters	
a_1	-0.0000002	A	653.5321517
a_2	0.0000384	B	-0.5924888
a_3	0.0012165	C	-4394.6011025
a_4	-0.3869002	D	-4946.0948988
a_5	-40.5282379		
a_6	7366.1539605		

Now by having the parameters of the stiffness model for F3.5C1.0, it is possible to determine stiffness at different temperatures and desired strain levels. As an example, Figure 5-4 shows the original stiffness-temperature curve and the one which is transferred to 0.1‰ strain level.

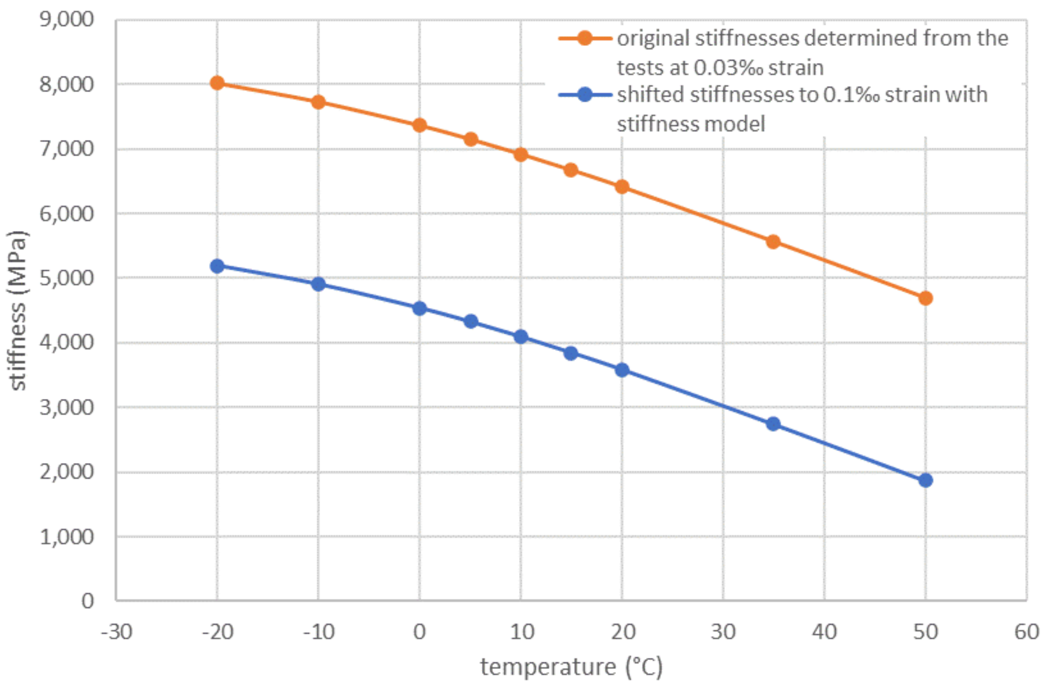


Figure 5-4: The original stiffness-temperature curve of the F3.5C1.0 mix (at around 0.03‰ strain level) and the transferred one to 0.1‰ strain level with the stiffness model

It is important to mention that when the material is stressed up to a certain level of strain, then for all situations which lead to a lower level of strain, there will be almost no strain dependency and there is only temperature dependency (see section 4.8.2 about the multi-round stiffness tests). To consider this behavior in practical design cases, the 2nd term of the general equation (the strain function) will be calculated for an already defined strain level and then will become a constant part. Further, only the temperature will affect the stiffness.

5.3 Shift factor to determine the fatigue life

As mentioned already the main issue with fatigue is to develop a meaningful relation between laboratory tests results and the field behavior. Factors such as the traffic lateral wander, crack propagation (material's response to failure propagation) and rate of loading (which provides rest / relaxing periods for the material), are the parameters that affect the difference between the laboratory fatigue life and the field performance. Shift factors are used to account for these differences. The last two parameters are influenced by the material's behavior to dynamic and cyclic loading. Considering that foamed bitumen stabilized mixes are not continuously bonded continuums, their response to the crack growth (extension) and resting periods may differ from conventional hot mixes (see section 4.12) and therefore, different shift factors are required to be considered for them. The second point is that the shift factor is dependent on the type of fatigue test and its boundary conditions too. Indirect tensile cyclic tests need higher amounts of shift factors compared to bending beam tests. Constant stress test types need higher shift factors compared to constant strain tests and tests with cyclic loads need higher shift factors than the ones with pulse loads (loads with a resting period). Test temperature is another factor that affects the fatigue life and therefore the shift factor. Considering mentioned points, determining a fatigue failure shift factor for foamed bitumen stabilized mixes, is a multivariable complicated task that requires reliable field behavior data too and is out of the scope of this research. To be able to perform the analytical design procedure, in the meantime a simple approach was used in this research to determine the shift factor for this material.

5.3.1 Description of the procedure

Two pavement sections with different thicknesses of F3.5C1.0 mix as the base layer (20 and 22 cm), and the same wearing and binder coarse thicknesses of 4 and 8 cm are considered as the study cases. They are referred to as the 20 and 22 cm sections in the text. Their life is determined by two different pavement design methods. The first one is the structural number (SN) method from AASHTO as an experimental-based model; with two different scenarios (normal and conservative). The second method is an analytical approach recommended by the Austroads for foamed bitumen stabilized mixes. Both methods with their references were explained in the literature review (see section 2.5). After determining the life based on these methods, they will be compared with the life determined from the laboratory-based fatigue function and then, a shift factor plus a safety factor will be determined.

For the asphaltic layers (wearing and binder courses of the two above-mentioned pavement sections) the calibration asphalts' data from RDO Asphalt 09 [201] are taken respectively. The required data for the F3.5C1.0 mix (volumetric, ITS, stiffness and fatigue) are taken from the results of different tests which are referred to in each step accordingly.

5.3.2 AASHTO 93 method

The main part of the AASHTO structural number method is to determine the structural layer coefficient of each layer of the pavement (Equation 2-21). Two scenarios are considered, the first scenario is based on equations that use each material's stiffness as the input to determine the layers coefficients (analytical-experimental based method). 2nd scenario is based on lower boundary amounts (a conservative scenario) to consider higher levels of uncertainties and not only the F3.5C1.0 of this research but a broader range of mixes in this class.

First scenario

There is an equation in the AASHTO 93 guide to determine the SN from the E modulus (page III-102, [161]):

$$\mathrm{SN}_{eff} = 0.0045D\sqrt[3]{E_P}$$ **Equation 5-5**

SN_{eff}, effective structural number of the pavement above subgrade (-)

D, total thickness of the pavement above subgrade (inches)

E_P, effective modulus of pavement layers above the subgrade (psi)

Considering that the SN is equal to the layer coefficient multiplied by the thickness, it is possible to determine the layer coefficient of asphaltic layers by having their stiffness modulus as under:

$$a = 0.0045\sqrt[3]{E}$$ **Equation 5-6**

a, structural layer coefficient of the layer (1/inches)

E, modulus of that layer at 20 °C (psi)

The stiffness of the calibrating asphalts as wearing and binder layers at 20 °C are 5,558 and 9,452 MPa (equal to 806,120 and 1,370,897 psi respectively); which results in structural layer coefficients of 0.42 (1/inch) for the wearing and 0.50 (1/inch) for the binder courses.

To determine the layer coefficient for the F3.5C1.0 mix combination, from the references which were mentioned in the literature review section (section 2.5.1), the equation from Khosravifar et al. [153] was selected for this purpose.

$$a = 0.0501 \times Ln\,(E) - 0.2636$$ **Equation 5-7**

a, structural layer coefficient of the foamed bitumen layer (1/cm)

E, stiffness modulus of the material at 25 °C, 10 Hz (MPa)

For further considerations, the worst-case state (combination of temperature and load) of the general stiffness model of the F3.5C1.0 mix (Table 5-1) is analyzed and utilized. This worst case is selected based on the maximum surface temperature of the pavement (47.5 °C based on temperature classes of the RDO Asphalt 09 [201]) and maximum allowable axle (11.5 ton single axle with 150 mm contact radius). The F3.5C1.0 layer is sub-layered into two layers. A try and error approach is applied to determine the stiffness of the layers. First, a tensile strain was assumed at the bottom of each sub-layer, then the stiffness was determined at that assumed strain and the calculated temperature in the middle of each sub-layer (with the RDO Asphalt 09 equation A 2.1 [201]). For the pavement, the tensile strains at the bottom of the two sub-layers are determined with BISAR and the results are compared with the assumed ones (which are used to determine the stiffness from the general stiffness model). If they are different, then the new amounts (from analysis results) are used to recalculate the stiffnesses and to reanalyze the pavement till the conversion is achieved. The final strain levels are then used as the base strains and then the stiffness of each layer is calculated at 25 °C and used as the input in Equation 5-7 to determine the structural layer coefficient for each of the 2 sub-layers. In the BISAR model, Poisson's ratio of the F3.5C1.0 material was taken as 0.28. Wearing and binder courses were sub-layered into 2 cm thicknesses and the stiffnesses were determined based on the temperature in the middle of each by using the calibration asphalt data and Poisson's ratio of 0.35. As the strain at the bottom of the F3.5C1.0 layers was desired, the subgrade and frost resistance layers were combined together for these cases. The stiffness of 120 MPa with the Poisson's ratio of 0.5 was selected for this layer (considering that the E_{v2} of minimum 120 MPa should be achieved on top of this layer).

After this analysis, for the determined horizontal strain level at the bottom of the F3.5C1.0 lower layer, was 0.2‰ and the stiffness at 25 °C, was calculated equal to 2,014 MPa which led to the layer coefficient of 0.118 (1/cm) or 0.3 (1/inch). For the upper layer, the strain level was determined as 0.1‰, stiffness 3,315 MPa (at 25 °C) and finally layer coefficient equal to 0.36 (1/inch).

Based on the amounts of structural layer coefficients of each layer, it is possible to calculate the total SN for the two pavement sections (with 20 and 22 cm of F3.5C1.0). For the section with 20 cm F3.5C1.0, SN of 4.8346 and for the 22 cm section, SN of 5.0945 were calculated. It is important to mention that the stiffnesses were calculated based on the 20 cm of F3.5C1.0 pavement case, therefore the same layer coefficients were used to calculate the SNs for both 20 and 22 cm and the only variable is the thickness. This will lead to a little lower life of the 22 cm section. Now it is possible to determine the life of the two sections with the AASHTO 93 equation (page II-32 of the guide [161]) by considering other input parameters of the equation (subgrade resilient modulus M_r = 120 MPa, standard normal deviate Z_r = -1.64 based on 95% reliability of the design according to table 4-1 page I-62 of the AASHTO 93, combined standard error of the input parameters S_0 = 0.4 and the difference between the initial and terminal design serviceability indexes ΔPSI = 2). As the life is based on 8 ton equivalent single axle load in this method, it is converted into 10 ton equivalent axle load by the power 4 damage function (factor of 2.44). It is 24.5 Million for the 20 cm and 35.8 Million for the 22 cm section.

Second scenario

As mentioned before this scenario considers lower amounts of structural layer coefficients for the materials to cover a wider range of qualities (lower than the calibration asphalts and the F3.5C1.0 results of this research) aimed to find out the bottom border of a safety factor besides the shift factor. For asphaltic layers, 0.42 (1/inch) and for the foamed bitumen stabilized layer, 0.3 (1/inch) as the lower amounts for each material type, from the first scenario, were selected as the layer coefficients for the calculation of SNs. The calculated life is 11.5 Million (equivalent 10-ton axles) for the 20 cm and 16.7 Million for the 22 cm section based on the AASHTO93 guideline equation with the mentioned parameters.

5.3.3 Austroads guide AP-T336-18 method

Austroads guide on "Design and Performance of Foamed Bitumen Stabilized Pavements" has a fatigue life transfer function for the foamed bitumen stabilized mixes with 95% of reliability level (Eq. 4 of page 51) [183]. The F3.5C1.0 material satisfies the characteristics of the mixes in this guide; therefore, this function was selected to determine the life of the two sections.

$$N = \left[\frac{6918(0.856V_b+1.08)}{\mu\varepsilon \times E^{0.36}}\right]^5$$ **Equation 5-8**

N, allowable number of repetitions of the load (-)

E, design modulus of the foamed bitumen mix (MPa)

V_b, percentage by volume of residual bitumen in the foamed bitumen mix (%)

$\mu\varepsilon$, tensile strain produced by the load (micro-strain)

The E is the design stiffness of the material. In this design guide, it is determined by the indirect tensile modulus test on soaked samples at 25 °C, then is justified for loading frequency (based on the design traffic speed) and layer temperature (weighted mean annual temperature). For this analysis, 20 °C was selected for the layer's temperature and all the stiffnesses were determined at this temperature. For the F3.5C1.0, the results of the worst-case were used again as inputs for strain amounts in the general stiffness model with 20 °C as the temperature input (resulting in 3,587 MPa for the upper F3.5C1.0 layer and 2,287 MPa for the lower). Then both pavement sections were analyzed with BISAR with the same model configurations as the worst-case model. The pavements were analyzed under a standard 10-ton axle load and with the stiffnesses which were calculated at 20 °C. The horizontal tensile strain at the bottom of the lower F3.5C1.0 sub-layer in each pavement section was determined as 0.1055‰ (105.5 micro strain) for the 20 cm section and 0.09591‰ (95.91 micro strain) for the 22 cm section. The volume of the bitumen in the mix was determined based on the volumetric parameters of the F3.5C1.0, equal to 7.7%. By putting these two parameters and the stiffness of the lower sub-layer (2,287 MPa) in the fatigue life transfer function (Equation 5-8), the life is 28.9 Million for the pavement with 20 cm foamed bitumen stabilized base and 46.6 Million for the section with 22 cm of F3.5C1.0 mix. Both are based on a 10-ton axle as the pavements were analyzed under this axle load.

5.3.4 Laboratory fatigue life and the shift factor

The tensile strain at the bottom of the lower layer of the F3.5C1.0 was used with its fatigue equation at 20 °C (10 Hz) to determine the fatigue life. The fatigue equation constants are C_1

= 4.1231 and C_2 = -4.6464 (section 4.11.3). Considering the above-calculated strains, the life of 20 cm section is 142,424 and the life of 22 cm one, 221,764 (equivalent 10-ton axles).

To determine the shift factor, the calculated lives resulting from the different applied methods are compared with the laboratory-based life. Table 5-2, summarizes all the lives together to give an overall view of the results.

Table 5-2: Life of the pavement sections with different design methods

Pavement section layers (cm)			Life calculation method			
Wearing	Binder	Base (F3.5C1.0)	AASHTO 93 (2nd scenario)	AASHTO 93 (1st scenario)	Austroads	Laboratory (20°C, 10Hz)
4	8	20	11,540,470	24,483,880	28,925,303	142,424
4	8	22	16,729,370	35,810,574	46,582,283	221,764

By dividing the life of each method to the laboratory-based life, some sort of shift factors can be defined. Table 5-3, shows the calculated ratios which are ranging from 75 to 210. Based on RDO Asphalt 09 [201], the shift factor for the hot mix asphalt base material is 1500. It is the result of 3 different parameters [96, 203, 204]: A factor of 70 to consider the crack extension, a factor of 20 to consider the rest periods (known also as healing) effect and a factor of 1.1 to take the lateral wander of the traffic into account. In the meantime, a safety factor of 2.1 for all traffic amounts is recommended too.

Table 5-3: Shift factors for foamed bitumen stabilized base layer, based on different design methods

Pavement section layers (cm)			Shift factor		
Wearing	Binder	Base (F3.5C1.0)	AASHTO 93 (2nd scenario)	AASHTO 93 (1st scenario)	Austroads
4	8	20	81	172	203
4	8	22	75	161	210

Considering the non-continuous nature of the bonds in F3.5C1.0 class mixes, it is acceptable to assume that the crack extension factor can be greater in these mixes compared to HMA bases. For the effect of the rest periods and the healing possibility, considering the failure mechanism and its growth, a lower amount compared to HMA seems reasonable in the meantime but both issues need more detailed research; therefore, the range of shift factor is smaller than 1500 and higher than 77. Considering the mentioned points and the analysis results (Table 5-3), at the mean time a shift factor of 200 is recommended for these types of material to transfer the laboratory fatigue life (based on indirect tensile cyclic tests at 20 °C and 10 Hz) to the field life. Comparing the AASHTO (1st scenario) and Austroads methods, a factor with a maximum of 1.3 is noticeable between the results. 2nd scenario results show a range for lower quality materials than the ones in this research. As mentioned before, the aim of considering this scenario was to define a lower boundary for the design.

A safety factor can be defined to consider different material qualities and design uncertainties. This factor can be between 1 to 2.5 and be selected based on the level of uncertainties in the input data. It is recommended to select an amount between 1 to 1.3 when the input data are determined from laboratory tests with low variability in results; and higher amounts (up to 2.5), when the results are with high variability.

5.4 Structural design with F3.5C1.0 material as the base layer

This section shows the results of the analytical structural design approach for F3.5C1.0 material by considering the before-mentioned points as the base. The designs were done for different capacities regarding the 7 loading classes from RStO 12 [202]. They are calculated based on a 10-ton equivalent single axle with single tires (15 cm radius of circular contact area). To perform the analysis and capacity control, the *ADtoPave*® 2018 software (Analyzing and design tool for pavements) from IDAV GmbH, was applied. It is a German-based software with different modules for analyzing different tests' results and designing the pavements. One of them is the design based on the analytical approach (at the meantime, the RDO Asphalt 09) which was already explained in this chapter.

5.4.1 Input parameters

For the stiffness of the F3.5C1.0 material, the strain level of 0.1‰ was selected and used as the input in the general stiffness model of the material (determined already, see Table 5-1). Then the stiffness-temperature equation's parameters were recalculated (polynomic with the order of 5) based on that. The *ADtoPave*® software has the possibility of using the stiffness master curve parameters of the material to consider the temperature dependency of the stiffness or to give it directly in the type of a polynomic equation with the order of 5. As the stiffness master curves of the foamed-cement mixes are based on one (and low) strain level, the polynomic input method can be utilized when the other strain levels are desired. Constant Poisson's ratio of 0.28 was selected for the layer. For determining the stiffness-temperature relation for the asphaltic layers (wearing, binder and the asphaltic base layers) in the form of polynomic five, the calibration asphalts' data from RDO Asphalt 09 [201] were used (see appendix F for the equation's parameters of each material). Constant Poisson's ratio of 0.35 was selected for asphaltic layers.

The fatigue life of the F3.5C1.0 layer was selected as the fatigue criteria. The fatigue function equation parameters of the material (based on the fatigue tests results @ 20 °C, 10 Hz) were used as input (C1 = 4.1231 as the constant and C2 = -4.6464, the power). A shift factor of 200 with a safety factor of 2.5 was used over the fatigue function. The reason for choosing such a high safety factor was to consider the lower boundary of the material qualities (both asphaltic and foamed stabilized). Therefore, this design results are relatively conservative and can be considered as a reference catalogue too.

Under the foamed-cement stabilized layer, a frost resistance layer (FSS) was considered with the E_{v2} of 120 MPa on top and material stiffnesses (E_s) of 120 to 150 MPa for loading classes of 1.0 to 100 million (BK1.0 to BK100). For the 0.3 million class, E_{v2} of 100 MPa with stiffnesses of 100 to 120 MPa was selected. For the subgrade, an E_{v2} of 45 MPa was considered for all the loading classes. 0.5 was taken as the Poisson's ratio of both layers. The freeze penetration depth was selected as 95 cm for the 100, 32 and 10 million loading classes, 90 cm for 3.2, 1.8 and 1.0 million classes and 80 cm for 0.3 million class. Zone 2 (RDO Asphalt 09) was selected for the temperature distribution probability. 100% bond was assumed between the asphaltic layers and the F3.5C1.0 layer. No bond was considered between the FSS layer with the F3.5C1.0 layer and with the subgrade. It was assumed that the same temperature gradient equation which is mentioned in RDO Asphalt 09 (equation A 2.1) [201] is also appliable for the foamed-cement stabilized material.

5.4.2 Design results and discussion

Based on the input data, for each of the loading classes, the thickness of the F3.5C1.0 layer was changed till the section's capacity became higher than the needed amount. In some loading classes, more than one design is possible. The thickness of the foamed-cement stabilized base was limited to 22 cm and if a higher thickness was necessary, other layers were changed or in case of the higher loading classes, a hot mix asphalt base layer was added too. Table 5-4, shows the design results for the different loading classes. It is important to mention that the thicknesses are determined to show the minimum needed thicknesses and maybe in some cases they are not practical in Germany in the meantime.

Table 5-4: Structural design results with F3.5C1.0 as the base layer for different traffic classes

Loading classes (Million 10 ton)		BK100	BK32	BK10	BK3.2	BK1.8	BK1.0	BK0.3
Pavement layers (cm)	Wearing (DS)	4	3	4	4	4	4	4
	Binder (BS)	8	6	8	6	-	-	-
	HMA Base (TS)	10	8	-	-	-	-	-
	FCSM Base	18	18	19	17	22	19	16
	Anti- frost (FSS)	55	60	64	63	64	67	60
Capacity (Million 10 ton)		103.9	32.6	11.9	3.99	2.4	1.06	0.345

To assess the effect of considering temperature dependency of the F3.5C1.0 mix (as the most representative FCSM with virgin parent material) on the structural design results, the capacity of the designed pavement sections (Table 5-4) were recalculated by considering a constant stiffness for the FCSM. The constant stiffness was calculated by using the general stiffness model and considering 20 °C as the temperature (10 Hz as the loading frequency) and 0.1‰ as the horizontal strain level. The calculated amount was 3,587 MPa. All other parameters remained the same as the ones explained already. It is important to notice that both models consider the dependency of the stiffness to the strain level. Table 5-5 shows the capacity of the sections based on the mentioned two models. It can be seen that applying a constant stiffness for the FCSM leads to underestimation of the capacity.

Table 5-5: Capacity of designed sections calculated with two stiffness models for the FCSM: constant (at 20 °C and 10 Hz) and temperature-dependent

FCSM's stiffness model	Loading classes (Million 10 ton)						
	BK100	BK32	BK10	BK3.2	BK1.8	BK1.0	BK0.3
Temperature dependent	103.9	32.6	11.9	3.99	2.4	1.06	0.345
Constant	103.8	32.2	11.5	3.86	2.2	0.96	0.320

The difference is not too much which is logical because of the lower temperature dependency of the F3.5C1.0 compared to HMA. It is important to keep in mind that this very small difference is valid for Germany's climate conditions and the same climate countries. In hotter climates (or in the case of slow-moving heavy traffic situations), the constant stiffness at 20 °C may lead to an overestimation of the pavement's capacity. An appropriate constant temperature is definable by considering the mean weighted yearly temperature of each country.

Considering the difference between the two stiffness models, it is possible to use the constant stiffness material model for FCSM as a simple analytical method for faster structural design purposes. As it is still important to consider the strain dependency of the stiffness, multi-step stiffness tests at an appropriate constant temperature and frequency (20 °C is appropriate for Germany with 10 Hz) are recommended to determine the stiffness. When such testing is not possible, a stiffness amount can be estimated from simple tests like ITS too.

It is also possible to combine the mentioned methods together and define a 3 level structural design approach which can be a good idea for a structural design guideline. The levels can be defined as:

<u>Level one</u> is a fast and simple method but with lower accuracy, based on an equivalency factor between FCSM and hot mix asphalt base material or a catalogue type table. Table 5-4 design results can be used as the base for this level. By comparing them to the thicknesses of the sections from RStO 12 (see table 1 row 1) [202], an equivalent factor of 1.4 to 1.5 can be defined to transfer the HMA base thickness to FCSM (1.5 for BK100 and BK32 and 1.4 for BK1.0 to BK10). It is to be noticed that these factors aren't the real equivalent factor between these two types of material as it is also affected by the construction type configurations.

<u>Level two</u> is a simple and relatively fast (lower amount of needed stiffness tests) method based on a constant stiffness amount for the foamed bitumen material. This method should be used with more caution in case of mixes with high contents of RAP as the parent material, especially when the RAP is still with active (not aged) bitumen.

<u>Level three</u> is a detailed analytical method that considers the temperature and also the strain level dependency of the material. The multi-step and stiffness-temperature tests will be applied to define the general stiffness model of the material.

It can be seen that by increasing the level of the design's accuracy, the complexity of the tests and calculations will increase too therefore, based on specific requirements and expectations of each project's phase it is possible to select an appropriate level.

5.5 Recommended design procedure for pavements with FCSM layers

This section summarizes the recommended procedure based on the findings of this research and the applied methods in this chapter for the analytical design of the pavements with FCSM as the base layer. The procedure is based on the level 3 design method. If the level 2 design is preferred, then step 3 can be ignored, and the step 2 results can be used to determine the input stiffness at 20 °C and the desired horizontal strain (0.1‰ is recommended).

Step one: It is recommended to prepare 24 specimens with the desired mix design, to be able to select a minimum of 20 specimens from them.

Step two: Performing multi-step stiffness tests on 5 specimens. The tests can be performed at any temperature but 20 °C and 10 Hz are recommended. The stiffness-strain level equation is determined from the tests' results (see section 5.2, step two). The specimens can later be used for ITS tests.

Step three: Taking the remaining 15 specimens for the stiffness tests at different temperatures and frequencies (3 for each of the 4 temperatures and keep 3 for reserve). It is recommended to perform the tests at low to medium horizontal tensile strain levels (0.03 to 0.05‰). The results are used to construct the stiffness master curve by selecting an appropriate Poisson's ratio. Then the stiffness master curve is used to define the stiffness-temperature equation (see section 5.2, step one).

Step four: Performing fatigue tests (at 20 °C and 10 Hz) on the 15 specimens used for stiffness tests (in step three). It is recommended to use 3 different stress levels and for each level 5 specimens to be tested. After that, the fatigue equation can be determined from the results.

Step five: By having the stiffness-strain level and stiffness-temperature equations the stiffness model of the material (general stiffness equation) can be determined based on the method explained in section 5.2. Now it is possible to determine the stiffness-temperature equation at any other strain level (which is selected by the designer) with the help of the general stiffness equation (in the meantime the amount of 0.1‰ is recommended).

Step six: Applying the stiffness-temperature (step five) and the fatigue equations (step four) as the input data for a mechanistic-empirical analytical design approach (like the one in RDO Asphalt 09 [201]). It is possible to consider the effect of moisture-related damages on the stiffness by a decreasing factor which can be determined based on the ratio of the stiffness after a moisture conditioning process to its unconditioned amount. It is recommended to use 200 as the shift factor and a relevant safety factor between 1 to 2.5 (based on the level of uncertainties in input data), until more research results on this issue become available. A linear elastic multi-layer computer program can be used for the modeling and analysis of the pavement.

It is important to mention that the design procedure is when the FCSM layer is applied as a bearing layer in a pavement system and fatigue can be considered as the material's primary failure mode. This procedure is a part of the pavement design and should be integrated into the whole design workflow. It means that based on each analytical pavement design, other criteria should also be controlled and satisfied. This procedure can be integrated into the RDO

Asphalt's analytical workflow (figure 2.1 of the RDO Asphalt 09 [201]) in the same place as the fatigue control for the hot mix asphalt base and the rest of the chart remains the same.

5.6 Summary

The aim of this chapter was to figure out if the German analytical pavement design approach (mentioned in RDO Asphalt 09) can be applied for pavements with FCSM as a base layer. The important input parameters and how to determine them for this type of material were discussed. It was shown that it is possible to use the analytical approach for the structural design of pavements with foamed bitumen stabilized material as the base layer. A design procedure was defined for this purpose which can be integrated into the RDO Asphalt procedure. The primary failure mode of the material was considered as fatigue. It is important to select the appropriate failure mode according to the contents of binding agents. It is possible to use the existing software in the market with some adjustments for this purpose. With the help of gained knowledge, it is now possible to perform different design scenarios by considering more than one layer of FCSM, with different parent materials and mix designs. The important point is to make the right decision on the selection of the primary failure mode of the material.

Chapter 6

6 Conclusions and Recommendations

Rising concerns on negative environmental effects of the construction / rehabilitation activities plus efficient use of the resources have increased the attention to sustainable technologies in the transportation sector. During the last years, different solutions and approaches have been introduced or further developed. Stabilization technology is one of these solutions. It can be adopted to produce materials with enhanced characteristics, lower production energy needs and higher assumption rates of recycled products. By utilizing foamed bitumen and cement as stabilizing binders, a composite product can be produced from different granular parent materials (named as foamed bitumen and cement stabilized material and abbreviated as FCSM) having a higher bearing capacity, lower moisture sensitivity with a balance between flexibility (resulted from bitumen) and rigidity (resulted from cement). It can be integrated into the pavement section with the aim of faster construction, higher share of recycled aggregates, lower production temperature and therefore lower emissions.

Understanding the material's behavior is necessary to be able to get the best out of mentioned advantages and deliver optimum characteristics based on the requirements of each project. Currently, the utilization of this material is limited in Germany. It is mainly due to the lack of national-level data and methods to evaluate the effect of different parameters on the behavior and performance of this material and later to be able to design it as a pavement layer. Among these parameters, the amount of two binding agents (foamed bitumen and cement) plays a big role in the characteristics of the resulting material. This research tried to take a deeper look into this material and assess the effect of these two binders on its mechanical and performance characteristics, and to integrate them into the existing national pavement design analytical approach. Based on the literature review and the existing national knowledge below questions were raised as the main tasks:

1- How to produce and prepare FCSM samples in the laboratory? According to the literature review, compaction and curing are the main issues in the production and preparation of the samples.
2- How to determine the physical parameters of FCSM samples?
3- How will FCSM respond to the load? how to determine mechanical parameters of FCSM for analysis of the material's response in a pavement model?
4- How does it perform under cyclic loading and what is the effect of binding agents' properties on that?
5- How to perform the structural design of pavements with FCSM as the base layer?

To address each of the above-mentioned questions, the approach was to apply as much as possible the existing and available knowledge (i.e., research, standards, guidelines and instructions) in Germany with the aim to figure out up to which level they are applicable for this material. In cases that justifications were needed, findings from the literature review were the base to select the best practice method by considering the availability of equipment and testing possibilities in the laboratory. Samples were produced with different combinations of bitumen and cement content but the same parent material and mix gradation. Five different mix combinations were tested with bitumen contents of 2.5, 3.5 and 4.5% and cement amount of 1% and with bitumen amount of 3.5% and cement contents of 2 and 3%. It was decided to use virgin aggregates to separate the effects of the recycled aggregates variability and in some cases the unknown interactions of old and new binders from the tests' results.

The results show and prove that it is possible to produce FCSM mixes with acceptable mechanical and performance characteristics and to utilize them as the base layer in pavement structures. Important points regarding mix production, sample preparation and determining the physical characteristics (density, maximum density and air voids contents) were pointed out. Material's response to static and dynamic load by varying the amounts of two binding agents were considered at different temperatures. The results were combined to develop a stiffness model of the material with the point of using that as an input for structural design. To assess the performance of the mixes, fatigue was selected aiming to figure out if it is possible to be considered as one of the failure modes of these mixes.

It is recommended to keep the cement content low (normally 1% and not more than 1.5% of the dry mass of aggregates). It is also possible to use lime instead of cement and it can be up to 2%. Based on the test's results of this research, bitumen amounts higher than 3% can lead to a resulting material that the fatigue can be taken as the primary failure mode.

By utilizing the findings of this research, it is now possible to produce FSCM mixes, prepare test specimens, calculate their main volumetric parameters and test them to determine the needed inputs for pavement structural design with the help of developed design approach in this research. It is now possible to design new pavement sections with different combinations of cold-produced mixes and later to compare them with conventional pavement sections.

The following sections are summarizing the main points and findings from this research and the ideas for future researches in this field.

6.1 Summary and recommendations

6.1.1 Mix production and sample preparation

Parent material gradation is one of the main parameters which should be considered to have a good pack of aggregates after compaction. In this research different fractions of aggregates were mixed to reach the gradation based on the Fuller method with n = 0.45 which led to a good aggregate pack. Special care should be taken on the amount of filler (< 0.075 mm) to satisfy the minimum needed amount which is 5%. If < 0.063 mm is considered as the filler size, it can be corrected accordingly. Coarser skeleton with lower fines leads to coarser mastic droplets and also poorer distribution which affects the scatter of the tests' results considerably.

Applying crushed aggregates will lead to the formation of an aggregate pack with higher shear resistance; therefore, it is recommended for practical cases to consider that during the mix design stage and try to produce mixes with different types of the parent material. The final mix can be then selected based on the results of the mix design stage tests or even after performing structural design for both scenarios and considering economical points.

It is important to use an appropriate bitumen for foaming. It is possible that some anti-foaming additives have been added to the bitumen during the refining process. To get a foam with a higher expansion ratio and half-life, some guidelines recommend using foaming agents. No foaming additives were applied in this research. The minimum expansion ratio of 10 times and half-life of 10 seconds recommended by M KRC [11] were applied for foaming mix design in this research.

Mixing moisture content can be selected between 60 to 80% of the OMC (optimum moisture content determined with modified Proctor test on parent material mixed with cement) which is affected by the fines fraction amount. This moisture content affects the distribution of the fine fraction in the mix and later the distribution of the bitumen. In this research, 75% was selected based on the fine amounts.

Standard Marshall sample size was selected and used for this project. The static compaction method of the M KRC [11], didn't lead to appropriate volumetric results, higher compressive load range is needed if it is desired to apply this compaction method. The compaction moisture equal to OMC (determined with modified Proctor test on parent material mixed with cement), is appropriate for compaction. The samples can be compacted by applying 75 blows to each side. Based on the author's experiences, if samples of diameter are desired (i.e., 150 mm), the vibratory hammer is recommended as the compaction method. The OMC in this case can be determined by using the same compaction method instead of the modified Proctor.

Two different curing methods were applied in this research named as standard method (based on M KRC [11]) and the fast curing procedure (72 hours at 40 °C). The fast curing method can be applied to decrease the production and preparation time. It is important that the used oven be able to extract the moisture out of the cabin and also to reach the constant state mass. If the requirements are not satisfied, then the tests' results of different batches may differ from each other.

6.1.2 Volumetric characteristics of the material

Bulk density can be determined by measuring the specimen's mass at the end of curing time (for fast curing at higher temperatures, after cooling to the room temperature) and by volume determined based on measured diameter and height of the specimens. If the water immersion method is applied to determine the volume, after measurement, the samples should again be put in the oven at 40 °C to reach the constant mass. The bulk densities determined with these two methods won't be the same and it is important to use only one method for the project.

Maximum density (known also as the zero air density) can be determined with the Rice method (see section 3.7) by applying the loose mixes which are cured in room conditions (20 ± 2 °C and 50 - 70% relative humidity) for 7 days. It is recommended to perform the test on 4 samples. Air voids content can be determined by having the bulk density and the maximum density. Maximum density is a parameter that is not affected by the compaction method and can be a robust index for controlling the compaction in the site or for producing specimens with defined amounts of air voids in the laboratory.

6.1.3 Stiffness characteristics

The indirect tensile cyclic test based on DIN EN 12697-26 annex F [108] or the German technical procedure TP Asphalt-StB Teil 26-2018 [187] can be used as a guide to perform the stiffness tests to determine the dynamic stiffness of the specimens. The appropriate horizontal strain range needs to be selected based on the aim of the test. In this research, the AL SP Asphalt 09 guideline [107] was applied to perform the stiffness tests at different temperatures, loading frequencies and horizontal strain levels.

Results of different stiffness tests showed the material's response is affected by temperature, loading rate, load amount (which determines the stress / strain level) and also its history. Therefore, if the stiffness at a certain temperature and frequency (normally 20 °C and 10 Hz) is desired, a multi-step stiffness test is recommended which is a stiffness test with horizontal strain sweep. Based on the results of these tests it is possible to see the effect of the horizontal strain level and select the stiffness based on an appropriate amount of horizontal strain. If the stiffness master curve is desired, until more research is done, based on the results of this research, the author recommends performing the tests at low to medium horizontal tensile strains (30 to 50 $\mu\varepsilon$) to be sure that the specimens have an acceptable level of damage.

Comparing the stiffness master curves of different mix combinations, revealed that the effect of cement is more than the effect of bitumen on the stiffness, therefore it is important to keep it low if flexibility is desired. Comparing the master curves with the reference hot mix showed lower temperature dependency of these mixes than the hot-made samples even with the same amount of bitumen.

To construct the stiffness master curves, the Arrhenius type equation (with 25,000 as the constant) was applied to determine the shift factors (α_T) and later the Sigmoidal function as regression curve (based on the AL SP Asphalt 09 guideline [107]). The results were acceptable and confirmed the findings of other researchers. More tests can be done to validate the results of the fitted curves for higher temperatures out of the tested ranges. The applicability of the TP Asphalt-StB Teil 26 [187] for these materials was considered and the two methods were compared together.

It is important to consider the effect of Poisson's ratio parameter during the stiffness calculation from the tests at different temperatures. In this research, two scenarios were compared. The first one was the asphaltic model, which considers temperature dependency, and the second scenario was to calculate the Poisson's ratio by the theory of elasticity approach by using the horizontal and vertical deformations of the specimens from indirect tensile strength tests data. It showed a relatively constant amount of 0.28 in all temperatures. The vertical deformations were taken on the loading piston, but it is recommended to use more precise methods for vertical deformation measurements.

Performing cyclic stiffness tests in high strain ranges up to 0.08‰, showed initiation of some micro-damages in weaker phase which influenced the amounts of dynamic stiffness, elastic modulus results and the stiffness master curves. It seems that it doesn't affect the ITS amounts and the fatigue tests' results as long as they were performed with higher initial strains than the one used in the cyclic stiffness tests

The loading direction can affect the result of stiffness because of the inherent heterogeneity of the material which is also common in HMA cylindrical samples too. It is recommended to test more samples to cover this issue in case of the multi-step test and to perform the tests in two different directions (90° rotation) in case of the stiffness tests for constructing the stiffness master curve. If rotation is used to measure the stiffness in two directions, the test's strain ranges in the first direction shouldn't be high to lead to inner damages which may affect the other direction's stiffness results.

During this research, two stiffness test types were introduced as multi-step and multi-round stiffness tests. Both provide a good insight into the stiffness behavior of FCSMs. Multi-step stiffness tests provide input data for structural design and also they can be performed on the cores drilled from pavements that are under service. In this case, they have the possibility to show the damage history in the material during its service period.

6.1.4 Fatigue characteristics

When foamed bitumen is used as the bituminous binder with cold mixes production method, the bituminous bonds in the resulted material will always be discontinued but based on the amount of cement, two different main scenarios can be defined. First, When the amount of cement is under the limit to form a bond microstructure (in the range of ≤ 1%). The material will have a weak (and non-continuous) bond state in the filler phase too. Under the action of stress, the bonds will damage preliminary through a fatigue type failure but because of a non-continuous state, the damage can't grow and propagate with the mechanism of propagation in continuum spaces (with the Paris law). In mixes with relatively high bitumen content (≥ 3%) the damage will be a combination of fatigue (primarily) and plastic deformation (secondary). For the mixes with lower bitumen amounts (in the range of BSM definition), it is relatively the same; but because of the higher effect non-continuity of the bituminous bonds (resulting from lower amounts of bitumen), plastic deformation (permanent deformation) is dominant as field observable damage mechanism. Second, when the cement content is enough to form a bond filler phase, the material will change to a relatively continuous state (can be assumed as a continuum) and therefore the failure mechanism is governed by the classical fatigue damage growth. It seems that 3% cement content can lead to the formation of the mentioned state. Cement contents between 1.5% to 3% will result in different responses which are not always the same and not easy to interpret because of the interaction of many parameters.

The indirect tensile cyclic fatigue test on cylindrical samples has been used in this research as the performance assessment test. Compared to the other fatigue test methods, it is easier to prepare the sample and perform the test. The main point is that this test reflects the response of the material to the cyclic load at the test's temperature and it is not always only a pure fatigue response. The energy ratio (ER) method was used to define the fatigue point and the related loading cycles to that for different mix combinations. The method is explained in TP Asphalt-StB Teil 24-2018 [188] and seems to be appropriate for FCSM too. Besides the ER vs. load cycles graph, the graph of cumulative horizontal plastic deformation vs. load cycles is a robust graph to detect the changing of the damage state during the test.

The fatigue equation with the form of $N = \mathrm{C}_1 {\varepsilon_{el,i}}^{C_2}$ was fitted to the fatigue life vs. initial elastic strain points (known as fatigue line). It is important to find the outlier pints correctly. Comparing the fatigue functions of the same mixes produced and tested in one laboratory at different times, showed a good level of repeatability and comparing the fatigue functions of the same mixes produced and tested in two different laboratories (with different equipment) also showed a good level of reproducibility. It is recommended to use more than 9 samples (15 samples, 5 at each stress level) for each round of fatigue tests to better cover the variability of the material which is its inherent characteristic.

Comparing the fatigue lines of different mix combinations showed an increase of the slope ($|C_2|$ parameter) when the mixes' stiffness rigidity increases (increase in the ratio of cement to bitumen). Higher stiffness shifts the fatigue line downward (decrease in the C_1 parameter).

Test temperature affects the fatigue behavior of the foamed bitumen mixes. Increasing the temperature from 5 °C to 10 and 20 °C causes to downward shift of the fatigue lines (in the case of using the initial elastic strain for the fatigue line equations). For structural design purposes, the results at 20 °C are recommended as they are more to the safe side. More research is needed to assess the effect of temperature on fatigue.

Based on the tests results of this research, bitumen amount ranges of 3 to 3.5% and cement ranges of 1 to1.5% are recommended to have a material with fatigue as the primary failure mode, when parent material is virgin aggregates' mix and contains less than 25% RAP.

6.1.5 Structural design

Both approaches of structural design (experimental and analytical) for pavements with foamed bitumen stabilized layers were introduced in the literature review. The analytical approach for the structural design of flexible pavements in Germany (RDO Asphalt 09 [201]), was applied as the base for the structural design of the pavements with FCSM base layers.

A method was developed which considers the temperature and stress / strain dependency of the stiffness together as the input for the pavement response analysis model. By this method, it is possible to shift the stiffness-temperature equation of the mix to any desired strain level. A simple method was introduced and used to determine the shift factor which transfers the laboratory determined life to the field life. Besides that, a range for the safety factors was recommended too.

The applicability of the developed method was demonstrated through a series of designs for different loading classes in Germany. Tests' results of the F3.5C1.0 mix data were used with a high safety factor amount. The results can be used as a reference catalogue too. Based on the findings, a 3 level structural design approach was defined with a practical instruction that can be used for the group of FCSMs with fatigue failure as their primary mode of failure.

6.2 Future works and researches

This research is the start of national researches on foamed bitumen stabilized / recycled mixes in Germany. The results can be considered as the base for future detailed and directional based researches on these types of mixes. Considering the results of this research, for future researches, three main directions are defined.

Direction one – Material science

1- *Detailed investigation on the failure mechanism of the foamed bitumen stabilized mixtures under indirect tensile loading mode*
Based on the tests' results and observations, different failure mechanisms were defined in this research. Detailed investigations on damage initiation, propagation and on possible healing, can be done with the help of different monitoring and scanning techniques on the sample during and after the tests. Methods like surface picture analysis or X-Ray scanning during the tests can be applied for this purpose. A deeper look at the behavior of the dissipated energy of each cycle and how it changes during different phases of the tests can be coupled with that to get a better understanding of the material's behavior and to be able to adjust the tests accordingly.

2- *An investigation on the parent material's properties effect on mechanical and performance characteristics of foamed bitumen stabilized mixes*
Parent material is believed to have the same importance as the binding agents on the resulted characteristics of the foamed bitumen stabilized mixes. In this research different properties of the parent material like their type, gradation-related parameters, surface and shape properties can be considered as variables. The main focus will be on the RAP aggregate aiming to identify how its different properties (especially the aging state of the bitumen) affect the resulting mix. The findings of this research will be applied to define methods and specifications for better preparation of the parent materials with the aim to optimize the resulting mix.

3- *Applying advanced computer modeling techniques for foamed bitumen stabilized mixes*
The research will use the results of advanced tests like the dynamic creep test in compression and indirect tensile modes for different combinations of bitumen and cement and at different temperatures to define the rheological model of the material. The resulting fundamental model of the material can then be integrated into a FEM code (finite element method) to model the material's behavior. It is also possible to combine it with DEM (district element method) too. These models provide the possibility of studying different complicated topics like the combined failure mods. This will also give the possibility of modeling the laboratory tests with the help of X-Ray scanning of the samples and to be able to better understand and interpret the response of the material to different variables. The results of this research will improve the process of testing and pavement structural design with foamed bitumen stabilized mixes.

4- *Assessing different rheological models for constructing the stiffness master curve of foamed bitumen mixes with validation of the appropriate method*
There are different methods for constructing the stiffness master curve. Mostly they differ in the rheological model which is used to calculate the shift factors (α_T). The research will assess these methods and try to find the appropriate one by validating the model-determined stiffnesses (at different temperatures and frequencies) with the ones which are from the stiffness tests' results of the same mixes, performed at that conditions.

Topics related to material's durability like the effect of moisture, low-temperature behavior and the freeze-thaw cycles, can be also considered under this research direction.

Direction two – Practical topics

1- *New construction / pavement types with foamed bitumen stabilized / cold recycled mixes*
In this research different combinations of the cold base layers will be used in pavement design to introduce new construction types. The aim is to decrease the volume of the hot mix asphalt and increase the share of cold mixes with virgin and recycled aggregates in different depths of the pavement.

2- *A performance-based mix design approach for foamed bitumen mixes*
The aim is to define a framework to integrate the mix design process with the structural design. The balanced mix design concept can be studied in this research by performing different pavement designs balancing the fatigue and permanent deformation resistance of the mix based on the amounts of bitumen and cement.

3- *Defining the acceptance limit for different test results of foamed bitumen mixes*
Foamed bitumen cold mixes are different from hot mix asphalt therefore different acceptance limits are needed to be defined for their test results assessments. This research will address this issue. Different numbers of samples can be tested to define the minimum needed one for each test considering the variation of the results. Ring tests can be planned and performed between different laboratories to evaluate the total possible variations in the results. The findings of this research can be integrated into testing guidelines.

4- *Potential applications of the foamed bitumen stabilized / recycled mixes as a base layer in railway tracks*
Foamed bitumen mixes can have properties ranging from enhanced unbound granular (having higher shear and moisture resistance) to bonded materials with more flexibility than cement stabilized bases. They have the in-situ production potential (which can be integrated into the existing railway rehabilitation machines) with higher application rates of recycled aggregates than the hot mix asphalt. These unique potentials can be considered in railway track upgrading projects as a good solution.

Direction three – Implementation

The behavior of foamed bitumen stabilized pavements under the accelerated pavement tests (APT).
Field test sections provide different possibilities to better understand the material’s response to load and temperature changes to be able to validate, justify and improve the mix and structural design stage parameters. They can be used to test and improve different destructive and non-destructive monitoring methods. These will pave the way for applying them in real test sections on the road network and efficient monitoring of them.

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8 Appendixes

A- Nyfoam 60, specifications

B- Foaming parameters

C- Optimum Moisture Content determination

D- Poisson's ratio amounts calculated from the ITS tests

E- Stiffness tests' results for stiffness master curves

F- Stiffness-temperature equations of the materials used for pavement design example

Appendixes

A- Nyfoam 60, specifications

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ISO 9001:2000 - COPRO

NYNAS

Nyfoam 60

Das Bindemittel Nyfoam 60 ist ein Spezialbitumen mit exzellenten und vorhersagbaren Schäumungseigenschaften. Es wird gemäß der EN 12591 Straßenbaubitumen und in Übereinstimmung mit den Nynas Spezifikationen hergestellt.

Straßenbaubitumen 50/70

Tabelle 1 : Spezifikationsinformationen

	Testbeschreibung	Methode	Einheit	Min	Max
Konsistenz bei mittlerer Verarbeitungstemperatur	Nadelpenetration bei 25 °C	EN 1426	mm/10	50	70
Konsistenz bei hoher Verarbeitungstemperatur	Erweichungspunkt RuK	EN 1427	°C	46	54
Beständigkeit gegen Verhärtung EN 12607-1	Massenänderung	EN 12607-1	%	-	0.50
	Verbleibende Penetration	EN 1426	%	50	-
	Anstieg des Erweichungspunktes	EN 1427	°C	-	11.0

Tabelle 2 : Zusätzliche Informationen

	Testbeschreibung	Methode	Einheit	Min	Max
Sicherheit und Umgang *	Minimale Pumptemperatur	-	°C	115	-
	Verarbeitungstemperatur**	-	°C	175	190
	Maximale Verarbeitungstemperatur	-	°C	-	200
	Flammpunkt (COC)	EN ISO 2592	°C	230	-
Technische Eigenschaften	Dichte bei 25°C	EN 15326	g/cm^3	1.024 [1]	
	Brechpunkt nach Fraaß	EN 12593	°C	-	-8
	Kinematische Viskosität bei 135 °C	EN 12595	mm^2/s	295	-
	Löslichkeit	EN 12592	%	99.0	-

* Für spezielle Informationsanfragen bezüglich Bitumenverarbeitung und Lagerung, sind Sie gebeten sich auf die Produktinformationsdatenblätter oder Sicherheitsdatenblätter zu beziehen.

** Typischer Verarbeitingstemperatur für Schäumbitumen

[1]typischer Wert

Unsere Produkte erfüllen die EN 12591 Spezifikationen für Strassenbaubitumen 50/70 und werden mit dem CE Kennzeichen versehen

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Sicherheitsdatenblatt auf www.nynas.com

B- Foaming parameters

Based on the foaming tests on different foaming water contents, the graphs of expansion ratio and half-life versus different foaming water content were drawn. By considering the limits of a minimum 10 times expansion ratio and 10 s half-life, it is possible to determine the range of acceptable foaming water. Based on the graphs, the amount of 3.3% was selected as it is equal to 12 l/h and is easier to set and control on the WLB 10S.

Table B-1: The foaming parameters of Nyfoam 60 measured at different foaming water contents

Bitumen Temperature (°C)		180			
Foaming Water (%)	Discharg (l/h)	Expansion Ratio		Half life (s)	
2	7.2	6	-	-	-
3	10.8	15	15	12	10
4	14.1	12	12	10	11
5	18	18	19	12	12
6	21	19	18	8	7

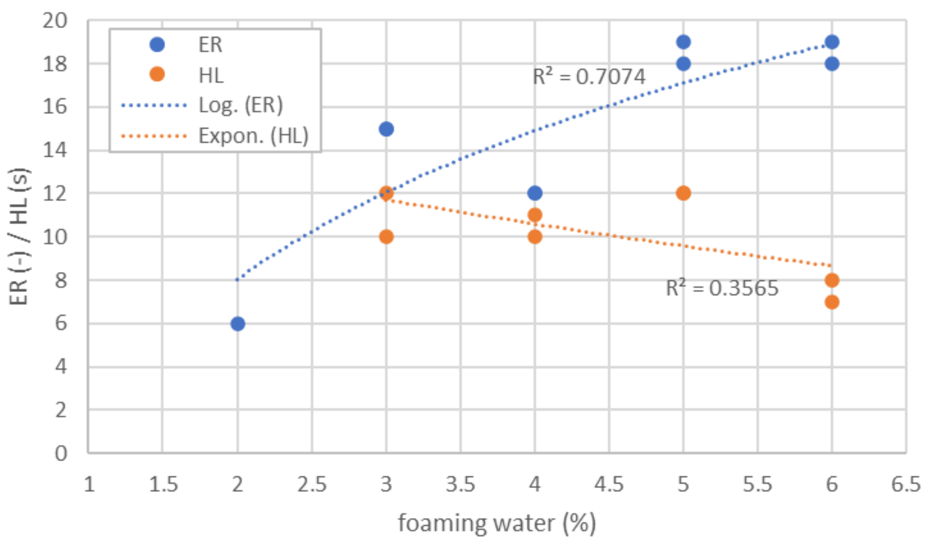

Figure B-1: Expansion ratio (ER) and half-life (HL) graph of the Nyfoam 60 at different foaming water contents

C- Optimum Moisture Content determination

First, the tests were performed with different cement contents. The water was calculated based on the weight of the dry aggregate mix plus the cement. It can be seen that by this method, the OMC remains almost the same at different cement contents.

Cement (%)	Moisture (%)	Dry Density (gr/cm^3)
1	3.5	2.3213
1	4.5	2.3856
1	5.5	2.4419
1	6.5	2.3844
OMC (%)	5.5	

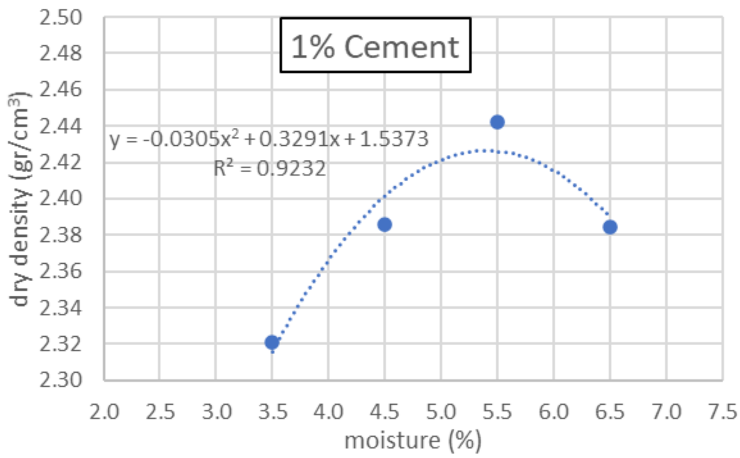

Figure C-1: Moisture-Density test results on the aggregate mix with 1% cement

Cement (%)	Moisture (%)	Dry Density (gr/cm^3)
2	4	2.3605
2	5	2.4071
2	6	2.461
2	7	2.3772
OMC (%)	5.6	

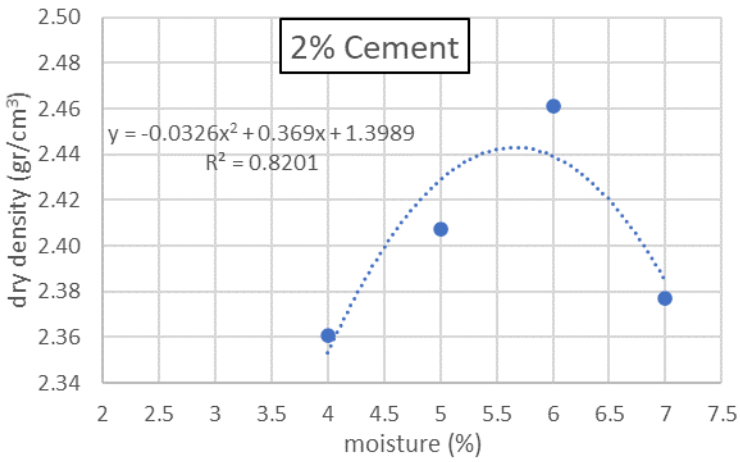

Figure C-2: Moisture-Density test results on the aggregate mix with 2% cement

Cement (%)	Moisture (%)	Dry Density (gr/cm^3)
3	3.5	2.3663
3	4.5	2.4087
3	5.5	2.4416
3	6.5	2.4102
OMC (%)	5.5	

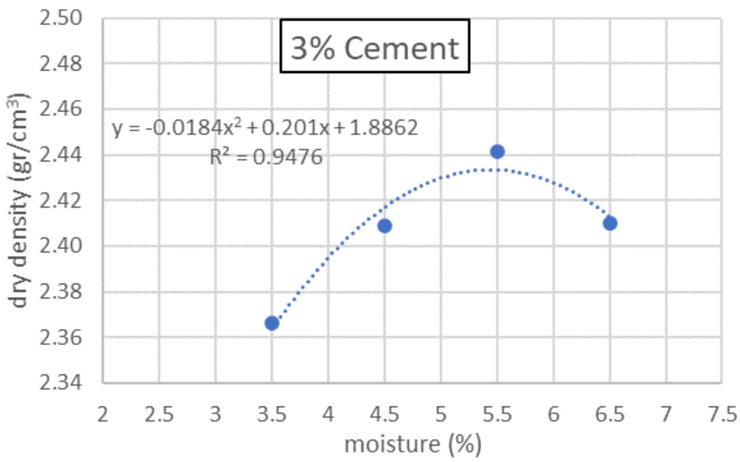

Figure C-3: Moisture-Density test results on the aggregate mix with 3% cement

After the above tests, the OMC was again determined only with 2% cement as double control. The results are displayed in the below figure.

Cement (%)	Moisture (%)	Dry Density (gr/cm^3)
2	3.5	2.3838
2	4.5	2.413
2	5.5	2.4716
2	6.5	2.4188
OMC (%)	5.4	

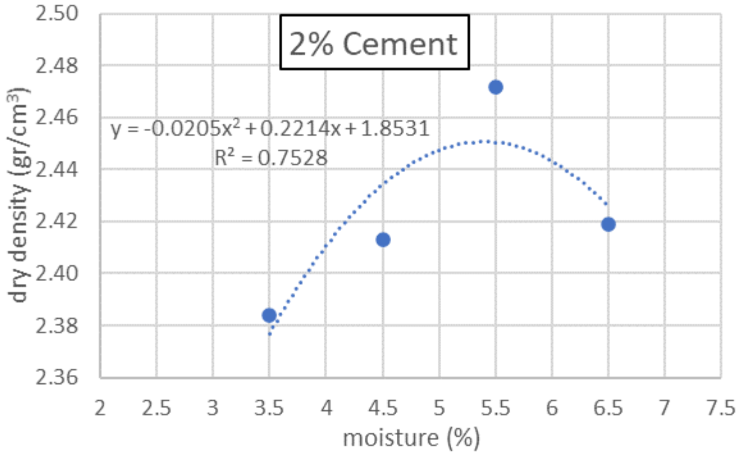

Figure C-4: Moisture-Density control test results on the aggregate mix with 2% cement

D- Poisson's ratio amounts calculated from the ITS tests

The formulation of the method is explained in section 2.3.3. and the calculated parameters are explained in section 4.4. Here the results of the calculated Poisson's ratio amounts at different temperatures and mix combinations are reported. To determine the Poisson's ratio, vertical and horizontal deformations of the samples during the ITS tests were used. Two different points were selected for this purpose. Based on these results, the amount of 0.28 was selected for the constant model Poisson's ratio.

Table D-1: Calculated Poisson's ratios of different mix combinations at different temperatures

Mix Combination		F2.5C1.0
Temerature (°C)	**2 µm H. Def.***	**45% ITS Defs.****
20	0.275	0.275
20	0.275	0.277
10	0.278	0.283
10	0.277	0.28
0	0.276	0.285
0	0.275	0.281
-10	0.275	0.3
-10	0.276	0.284

Mix Combination		F3.5C1.0
Temerature (°C)	**2 µm H. Def.***	**45% ITS Defs.****
20	0.274	0.277
20	0.276	0.278
10	0.27	0.282
10	0.27	0.283
0	0.275	0.308
0	0.275	0.284
-10	0.275	0.306
-10	0.275	0.278

Mix Combination		F4.5C1.0
Temerature (°C)	**2 µm H. Def.***	**45% ITS Defs.****
20	0.275	0.279
20	0.275	0.28
10	0.277	0.285
10	0.277	0.284
0	0.275	0.284
0	0.275	0.282
-10	0.275	0.278
-10	0.276	0.283

Mix Combination		F3.5C2.0
Temerature (°C)	**2 µm H. Def.***	**45% ITS Defs.****
20	0.273	0.273
20	0.274	0.277
10	0.278	0.278
10	0.275	0.278
0	0.275	0.275
0	0.276	0.28
-10	0.275	0.277
-10	0.275	0.277

Mix Combination		F3.5C3.0
Temerature (°C)	**2 µm H. Def.***	**45% ITS Defs.****
20	0.275	0.276
20	0.276	0.278
10	0.278	0.283
10	0.275	0.279
0	0.275	0.276
0	0.274	0.275
-10	0.275	0.277
-10	0.27	0.28

* Based on the 2 µm horizontal and the related vertical deformations.

** Based on the horizontal and vertical deformations related to 45% of the maximum load (or 45% of ITS).

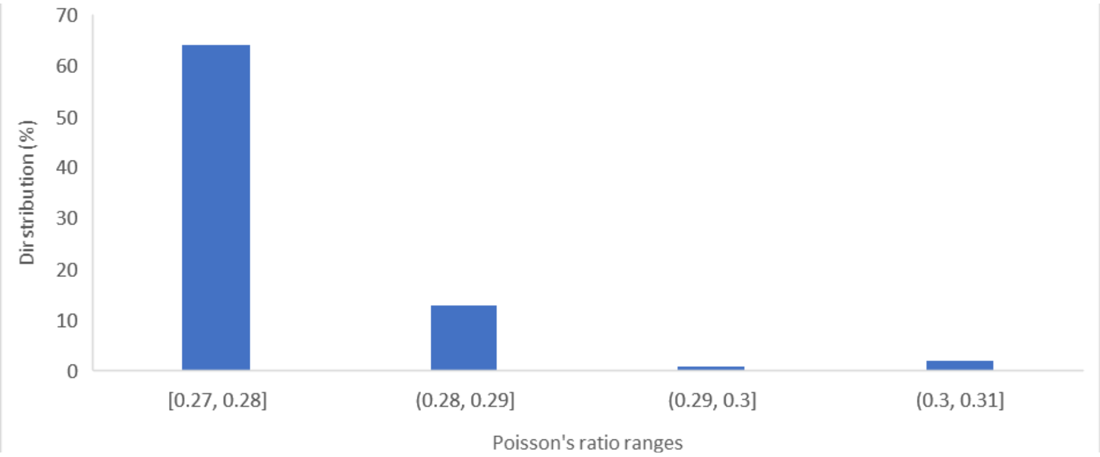

Figure D-1: Distribution of the Poisson's ratio amounts

E- Stiffness tests' results for stiffness master curves

Table E-1: Stiffness test's results of F2.5C1.0 mix

F2.5C1.0 mix combination				As. Model Poisson's ratio		Constant Poisson's ratio	
Prüftemperatur (°C)	Specimen Nr.	Frequency (Hz)	Upperstress (MPa)	ε_{el} (‰)	Stiffness (Mpa)	ε_{el} (‰)	Stiffness (Mpa)
-10	2-2	10	0.15	0.0168	9,671	0.0164	11,949
		5	0.15	0.0173	9,013	0.0169	11,136
		1	0.14	0.0178	8,946	0.0174	11,053
		0.1	0.14	0.0174	8,680	0.017	10,725
	2-7	10	0.16	0.0167	10,277	0.0164	12,698
		5	0.15	0.0174	9,662	0.017	11,937
		1	0.15	0.0178	9,297	0.0174	11,487
		0.1	0.14	0.0192	8,234	0.0188	10,173
	2-8	10	0.13	0.0183	7,888	0.0169	9,746
		5	0.13	0.0178	7,750	0.0163	9,576
		1	0.12	0.0165	7,779	0.0161	9,611
		0.1	0.11	0.0158	7,128	0.0155	8,808
0	2-4	10	0.11	0.0156	7,319	0.0153	8,584
		5	0.11	0.0155	6,928	0.0153	8,126
		1	0.10	0.0157	6,516	0.0155	7,642
		0.1	0.10	0.0159	5,927	0.0157	6,951
	2-6	10	0.12	0.0163	8,022	0.0161	9,408
		5	0.12	0.0171	7,657	0.0168	8,980
		1	0.12	0.0174	7,250	0.0171	8,503
		0.1	0.11	0.0171	6,914	0.0168	8,109
	2-10	10	0.11	0.0163	6,610	0.0161	7,752
		5	0.11	0.0169	6,384	0.0167	7,487
		1	0.10	0.0166	6,185	0.0163	7,253
		0.1	0.10	0.018	5,710	0.0177	6,697
10	2-4	10	0.17	0.0309	7,146	0.0307	7,709
		5	0.16	0.0315	6,557	0.0313	7,074
		1	0.15	0.0321	6,116	0.0319	6,598
		0.1	0.14	0.0328	5,456	0.0326	5,886
	2-7	10	0.16	0.0234	8,548	0.0241	9,222
		5	0.15	0.0235	8,226	0.0233	8,875
		1	0.14	0.0233	7,658	0.0231	8,262
		0.1	0.13	0.0242	6,699	0.024	7,227
	2-8	10	0.12	0.0178	7,840	0.0177	8,458
		5	0.11	0.0171	7,283	0.017	7,858
		1	0.11	0.0171	6,920	0.017	7,465
		0.1	0.10	0.0174	6,346	0.0173	6,846
20	2-1 (1)	10	0.10	0.0243	5,019	0.0243	4,858
		5	0.10	0.0265	4,641	0.0266	4,492
		1	0.10	0.0304	4,191	0.0305	4,056
		0.1	0.10	0.033	3,870	0.0331	3,745
	2-3 (2)	10	0.13	0.0369	4,563	0.037	4,416
		5	0.11	0.0321	4,308	0.0322	4,170
		1	0.11	0.0354	3,966	0.0355	3,839
		0.1	0.10	0.0363	3,352	0.0365	3,244
	2-5 (2)	10	0.13	0.0404	4,167	0.0405	4,033
		5	0.11	0.0351	3,920	0.0353	3,794
		1	0.10	0.0341	3,561	0.0342	3,446
		0.1	0.09	0.0307	3,346	0.0308	3,238
	were not used in the construction of the stiffness master curve						

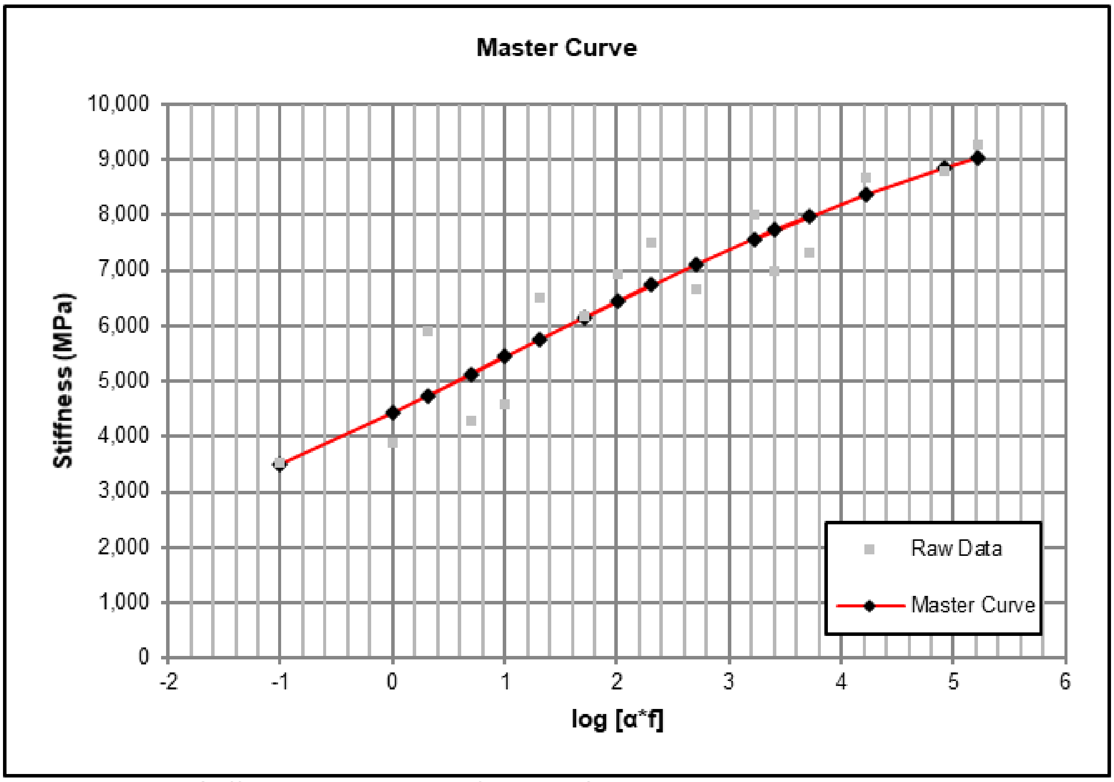

Figure E-1: Stiffness master curve of the F2.5C1.0 with asphaltic Poisson's ratio model

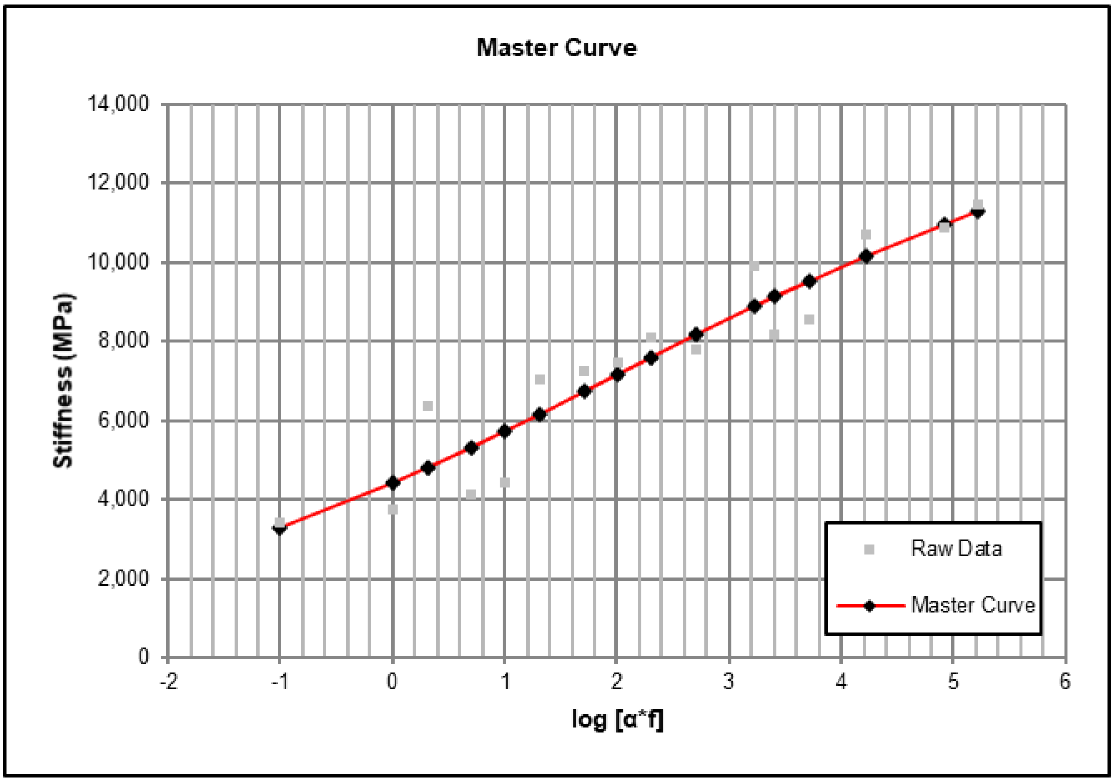

Figure E-2: Stiffness master curve of the F2.5C1.0 with constant (= 0.28) Poisson's ratio model

Table E-2: Stiffness test's results of F3.5C1.0 mix

F3.5C1.0 mix combination				As. Model Poisson's ratio		Constant Poisson's ratio	
Prüftemperatur (°C)	Specimen Nr.	Frequency (Hz)	Upperstress (MPa)	ε_{el} (‰)	Stiffness (Mpa)	ε_{el} (‰)	Stiffness (Mpa)
-10	3-1	10	0.16	0.0184	9,846	0.018	12,165
		5	0.16	0.0199	9,197	0.0195	11,364
		1	0.16	0.021	9,045	0.0205	11,175
		0.1	0.16	0.0216	8,396	0.0212	10,374
	3-2	10	0.13	0.0212	6,478	0.0208	8,003
		5	0.13	0.0206	6,386	0.0202	7,890
		1	0.12	0.0208	6,158	0.0203	7,609
		0.1	0.12	0.0209	5,779	0.0204	7,140
	3-3	10	0.14	0.0204	7,404	0.0199	9,148
		5	0.14	0.0203	7,169	0.0199	8,857
		1	0.13	0.0198	7,287	0.0194	9,003
		0.1	0.13	0.0189	7,609	0.0185	9,402
0	3-7	10	0.15	0.0229	7,334	0.0225	8,602
		5	0.14	0.0226	7,119	0.0222	8,349
		1	0.14	0.0243	6,553	0.0239	7,685
		0.1	0.13	0.0252	5,971	0.0248	7,003
	3-8	10	0.15	0.0246	7,059	0.0242	8,279
		5	0.14	0.0236	7,075	0.0232	8,298
		1	0.14	0.0223	7,045	0.022	8,262
		0.1	0.13	0.0244	6,157	0.024	7,221
	3-9	10	0.14	0.0215	7,469	0.0212	8,760
		5	0.14	0.0211	7,374	0.0208	8,649
		1	0.13	0.0214	6,984	0.021	8,190
		0.1	0.13	0.0215	6,627	0.0211	7,772
10	3-4	10	0.13	0.0241	6,282	0.0239	6,778
		5	0.12	0.0221	6,131	0.0219	6,614
		1	0.11	0.0218	5,708	0.0217	6,158
		0.1	0.10	0.0197	5,612	0.0195	6,055
	3-5	10	0.13	0.0197	7,715	0.0195	8,324
		5	0.12	0.0191	7,298	0.0189	7,873
		1	0.12	0.0208	6,792	0.0206	7,328
		0.1	0.11	0.0202	6,320	0.02	6,818
	3-6	10	0.13	0.0204	7,857	0.0203	8,477
		5	0.13	0.0216	7,099	0.0214	7,659
		1	0.12	0.0228	6,346	0.0226	6,846
		0.1	0.12	0.0256	5,320	0.0254	5,740
20	3-1	10	0.13	0.0252	7,981	0.0231	7,725
		5	0.13	0.0212	7,618	0.0213	7,374
		1	0.12	0.0222	7,209	0.0222	6,978
		0.1	0.12	0.0234	6,332	0.0235	6,128
	3-2	10	0.11	0.021	6,347	0.0211	6,143
		5	0.11	0.022	5,720	0.022	5,536
		1	0.10	0.0232	5,257	0.0233	5,088
		0.1	0.10	0.0232	4,838	0.0233	4,682
	3-3	10	0.11	0.0193	6,885	0.0193	6,664
		5	0.11	0.0198	6,297	0.0199	6,094
		1	0.10	0.0216	5,679	0.0216	5,496
		0.1	0.10	0.0218	5,040	0.0219	4,878
	were not used in the construction of the stiffness master curve						

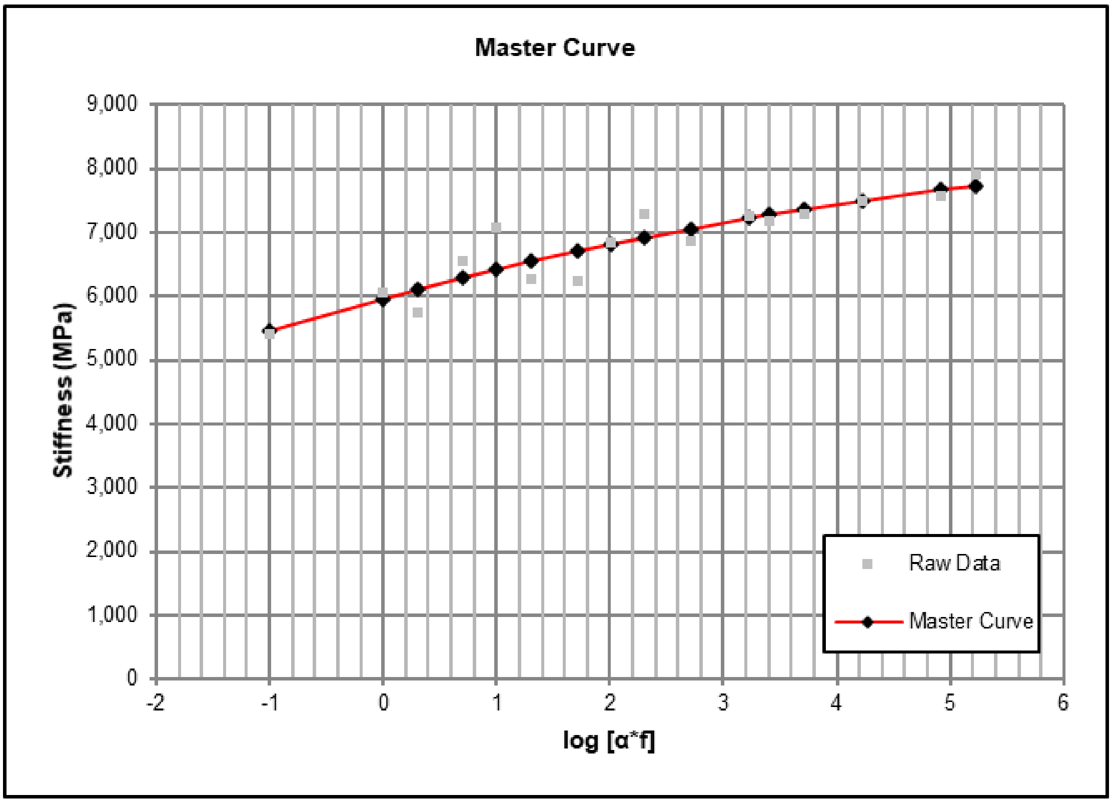

Figure E-3: Stiffness master curve of the F3.5C1.0 with asphaltic Poisson's ratio model

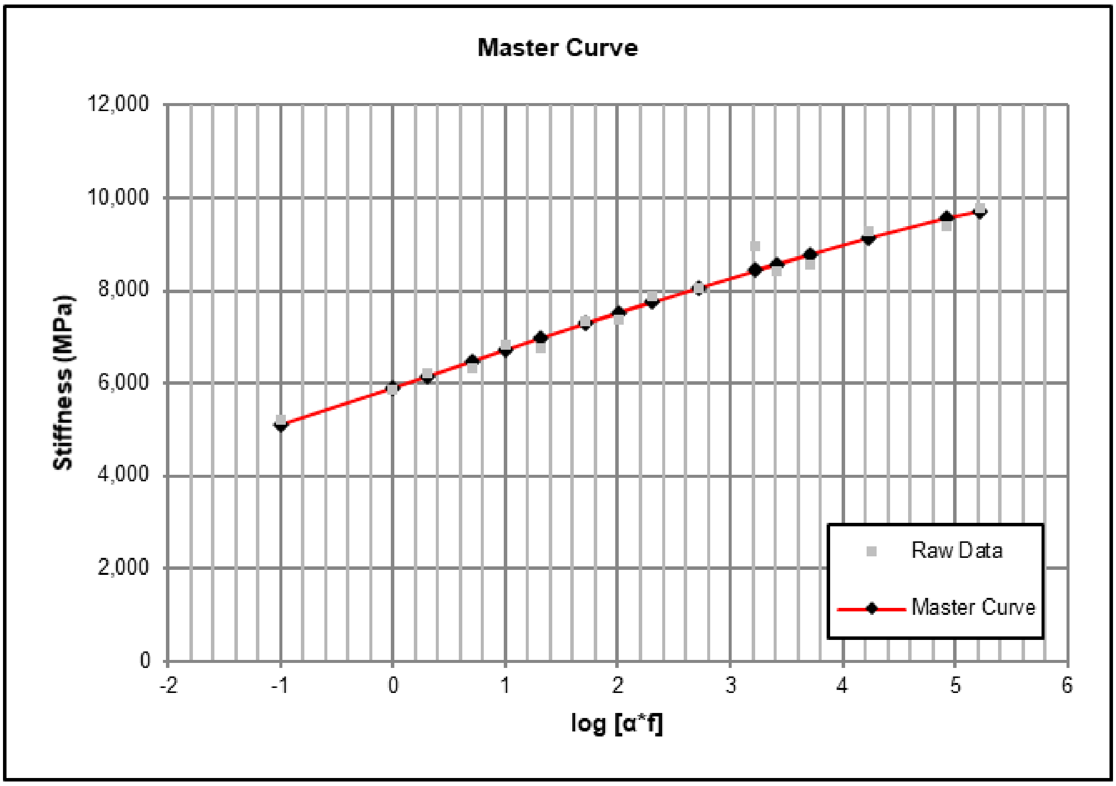

Figure E-4: Stiffness master curve of the F2.5C1.0 with constant (= 0.28) Poisson's ratio model

Table E-3: Stiffness test's results of F3.5C1.0-F mix

F3.5C1.0-F mix combination				As. Model Poisson's ratio		Constant Poisson's ratio	
Prüftemperatur (°C)	Specimen Nr.	Frequency (Hz)	Upperstress (MPa)	ε_{el} (‰)	Stiffness (Mpa)	ε_{el} (‰)	Stiffness (Mpa)
-10	12-1	10	0.25	0.0254	12,435	0.0248	15,364
		5	0.25	0.026	12,398	0.0254	15,319
		1	0.25	0.0274	11,814	0.0268	14,597
		0.1	0.25	0.0305	10,446	0.0298	12,906
	12-6	10	0.22	0.0255	10,481	0.025	12,950
		5	0.22	0.0264	10,369	0.0258	12,812
		1	0.22	0.0265	10,200	0.0259	12,602
		0.1	0.22	0.0296	9,108	0.0289	11,254
	12-8	10	0.20	0.0251	9,285	0.0245	11,472
		5	0.20	0.0257	9,195	0.0251	11,361
		1	0.20	0.027	8,573	0.0264	10,593
		0.1	0.20	0.0324	7,557	0.0317	9,337
0	12-5	10	0.19	0.0218	10,872	0.0215	12,751
		5	0.19	0.022	10,990	0.0217	12,890
		1	0.19	0.0234	10,474	0.023	12,284
		0.1	0.19	0.0257	9,530	0.0252	11,177
	12-7	10	0.18	0.0224	9,668	0.022	11,339
		5	0.18	0.0209	10,524	0.0205	12,343
		1	0.18	0.0215	10,226	0.0211	11,993
		0.1	0.18	0.0241	9,192	0.0237	10,781
	12-10	10	0.15	0.0212	7,861	0.0209	9,220
		5	0.15	0.0232	7,220	0.0228	8,468
		1	0.14	0.0241	6,710	0.0237	7,869
		0.1	0.14	0.0273	5,753	0.0269	6,747
10	12-2	10	0.17	0.0206	10,222	0.0205	11,028
		5	0.17	0.0212	10,096	0.0211	10,892
		1	0.17	0.0248	8,943	0.0246	9,648
		0.1	0.16	0.028	7,250	0.0278	7,821
	12-4	10	0.17	0.0218	9,771	0.0216	10,542
		5	0.17	0.0231	9,320	0.0229	10,055
		1	0.17	0.0258	8,272	0.0256	8,925
		0.1	0.15	0.028	6,934	0.0278	7,481
	12-6	10	0.16	0.0279	6,985	0.0277	7,535
		5	0.15	0.0261	6,914	0.0259	7,460
		1	0.14	0.0271	6,313	0.0269	6,811
		0.1	0.13	0.0267	5,694	0.0265	6,143
20	12-1	10	0.16	0.0243	9,179	0.0244	8,884
		5	0.16	0.0228	9,903	0.0229	9,584
		1	0.16	0.0259	8,792	0.026	8,509
		0.1	0.15	0.0273	7,779	0.0274	7,529
	12-3	10					
		5					
		1					
		0.1					
	12-8	10	0.20	0.0371	8,294	0.0372	8,027
		5	0.20	0.0364	8,277	0.0366	8,011
		1	0.19	0.0401	7,068	0.0402	6,841
		0.1	0.18	0.0492	5,437	0.0493	5,263
	were not used in the construction of the stiffness master curve						

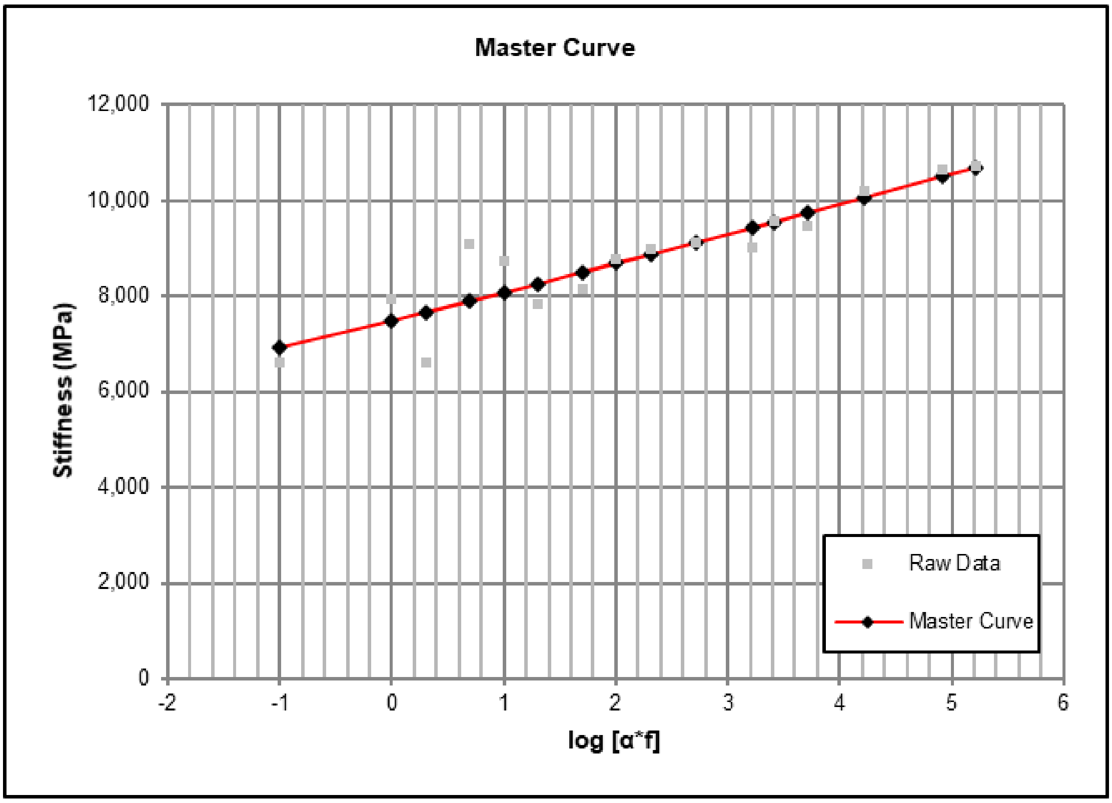

Figure E-5: Stiffness master curve of the F3.5C1.0-F with asphaltic Poisson's ratio model

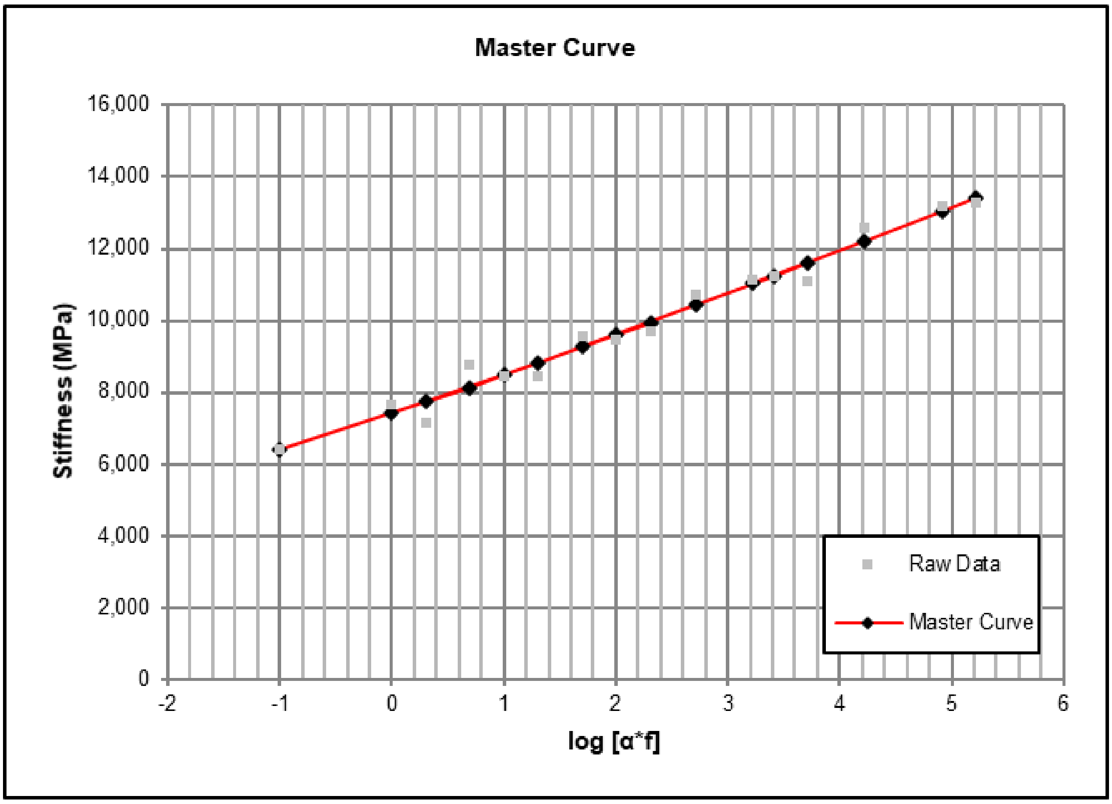

Figure E-6: Stiffness master curve of the F3.5C1.0-F with constant (= 0.28) Poisson's ratio model

Table E-4: Stiffness test's results of F4.5C1.0 mix

F4.5C1.0 mix combination				As. Model Poisson's ratio		Constant Poisson's ratio	
Prüftemperatur (°C)	Specimen Nr.	Frequency (Hz)	Upperstress (MPa)	ε_{el} (‰)	Stiffness (Mpa)	ε_{el} (‰)	Stiffness (Mpa)
-10	6-1	10	0.16	0.0238	7,832	0.0233	9,676
		5	0.16	0.024	7,621	0.0235	9,417
		1	0.16	0.025	7,279	0.0244	8,993
		0.1	0.15	0.0261	6,661	0.0255	8,230
	6-2	10	0.19	0.021	11,006	0.0206	13,599
		5	0.19	0.0214	10,998	0.021	13,589
		1	0.19	0.0212	10,959	0.0208	13,540
		0.1	0.19	0.0237	9,914	0.0231	12,249
	6-3	10	0.16	0.0238	7,670	0.0233	9,477
		5	0.16	0.0237	7,486	0.0232	9,250
		1	0.16	0.0264	7,047	0.0259	8,707
		0.1	0.15	0.0252	6,534	0.0247	8,073
0	6-7	10	0.15	0.0254	6,877	0.025	8,065
		5	0.14	0.0244	6,528	0.024	7,656
		1	0.13	0.0256	5,915	0.0252	6,937
		0.1	0.12	0.0257	5,212	0.0253	6,112
	6-8	10	0.15	0.0221	8,192	0.0217	9,607
		5	0.15	0.0215	8,002	0.0212	9,385
		1	0.14	0.0222	7,385	0.0218	8,661
		0.1	0.14	0.0246	6,238	0.0242	7,316
	6-11	10	0.17	0.0224	9,021	0.022	10,580
		5	0.17	0.023	8,931	0.0226	10,474
		1	0.16	0.0229	8,560	0.0225	10,039
		0.1	0.16	0.0267	7,416	0.0263	8,697
10	6-4	10	0.20	0.0224	12,298	0.0222	13,268
		5	0.19	0.0224	11,343	0.0222	12,237
		1	0.17	0.0214	10,659	0.0213	11,500
		0.1	0.16	0.0215	9,536	0.0213	10,288
	6-5	10	0.16	0.0203	9,840	0.0201	10,616
		5	0.15	0.0203	9,555	0.0201	10,309
		1	0.15	0.0206	9,053	0.0205	9,767
		0.1	0.14	0.0223	8,017	0.0222	8,649
	6-6	10	0.14	0.0209	8,497	0.0207	9,167
		5	0.14	0.0204	8,420	0.0203	9,084
		1	0.14	0.0217	7,844	0.0215	8,462
		0.1	0.13	0.0233	6,940	0.0231	7,488
20	6-1	10	0.16	0.0232	10,044	0.0233	9,721
		5	0.16	0.0267	8,874	0.0268	8,589
		1	0.16	0.0287	8,154	0.0288	7,892
		0.1	0.16	0.0343	6,598	0.0344	6,386
	6-2	10	0.16	0.0248	9,467	0.0249	9,163
		5	0.16	0.0284	8,101	0.0285	7,841
		1	0.15	0.0279	7,672	0.0279	7,425
		0.1	0.14	0.0269	7,173	0.027	6,943
	6-3	10	0.14	0.0228	8,166	0.0228	7,903
		5	0.14	0.0244	7,338	0.0244	7,102
		1	0.13	0.0263	6,795	0.0263	6,577
		0.1	0.12	0.0253	6,286	0.0254	6,084
	were not used in the construction of the stiffness master curve						

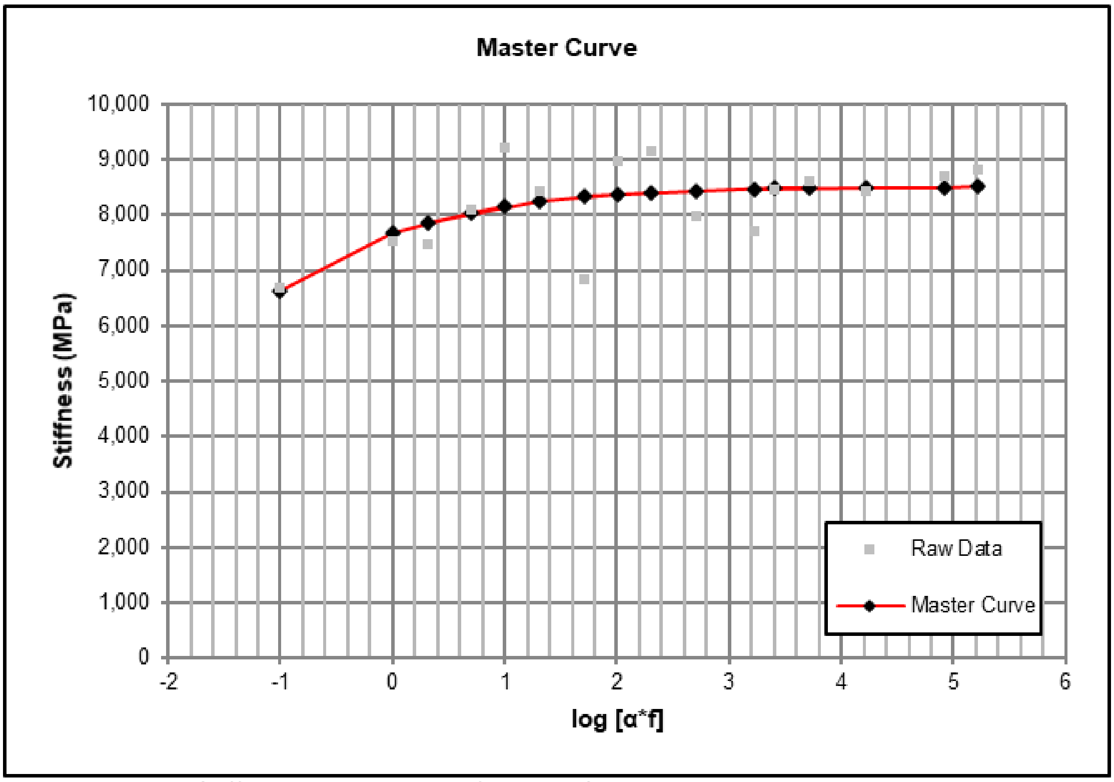

Figure E-7: Stiffness master curve of the F4.5C1.0 with asphaltic Poisson's ratio model

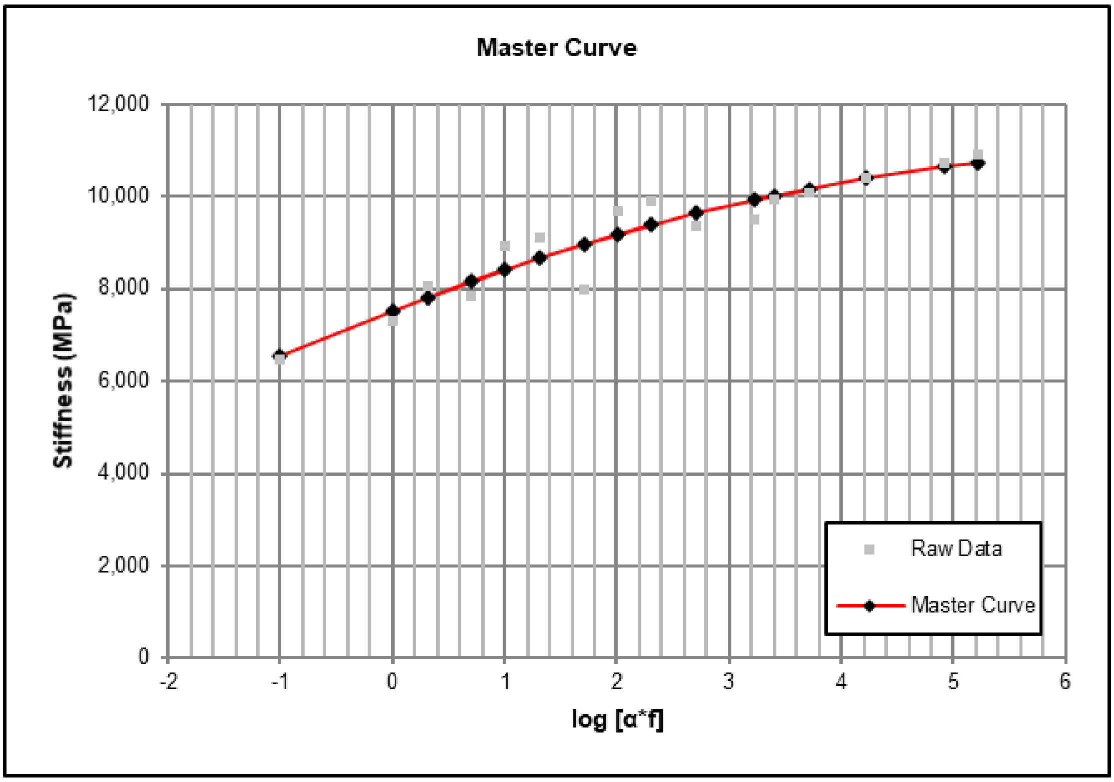

Figure E-8: Stiffness master curve of the F4.5C1.0 with constant (= 0.28) Poisson's ratio model

Table E-5: Stiffness test's results of F3.5C2.0 mix

F3.5C2.0 mix combination				As. Model Poisson's ratio		Constant Poisson's ratio	
Prüftemperatur (°C)	Specimen Nr.	Frequency (Hz)	Upperstress (MPa)	ε_{el} (‰)	Stiffness (Mpa)	ε_{el} (‰)	Stiffness (Mpa)
-10	7-2	10	0.35	0.0343	13,609	0.0335	16,814
		5	0.35	0.0352	13,589	0.0344	16,790
		1	0.35	0.0358	13,287	0.035	16,417
		0.1	0.35	0.0384	12,451	0.0375	15,384
	7-4	10	0.37	0.0352	14,324	0.0344	17,699
		5	0.37	0.0359	14,301	0.0351	17,670
		1	0.37	0.0365	13,865	0.0357	17,131
		0.1	0.37	0.0386	13,168	0.0377	16,292
	7-10	10	0.30	0.0332	11,975	0.0324	14,796
		5	0.30	0.0347	11,644	0.0339	14,386
		1	0.30	0.0358	11,134	0.035	13,757
		0.1	0.30	0.0384	10,385	0.0376	12,831
0	7-3	10	0.45	0.0356	18,496	0.035	21,692
		5	0.45	0.036	18,600	0.0355	21,815
		1	0.45	0.0371	17,631	0.0365	20,677
		0.1	0.45	0.0409	16,162	0.0402	18,955
	7-6	10	0.23	0.0199	15,356	0.01195	18,009
		5	0.23	0.0196	15,859	0.0193	18,559
		1	0.23	0.0199	15,529	0.0195	18,213
		0.1	0.23	0.0207	14,966	0.0204	17,553
	7-9	10	0.25	0.0208	16,050	0.0204	18,823
		5	0.25	0.0208	16,309	0.0205	19,128
		1	0.25	0.021	16,066	0.0206	18,842
		0.1	0.25	0.0216	15,769	0.0212	18,494
10	7-1	10	0.21	0.0212	13,547	0.021	14,616
		5	0.21	0.0217	13,487	0.0211	14,626
		1	0.21	0.0224	12,917	0.0222	13,936
		0.1	0.21	0.0239	12,291	0.0237	13,260
	7-4	10	0.30	0.0301	15,074	0.0299	16,263
		5	0.30	0.0315	14,681	0.0312	15,839
		1	0.30	0.0316	14,204	0.0313	15,324
		0.1	0.30	0.0329	13,550	0.0326	14,619
	7-9	10					
		5					
		1					
		0.1					
20	7-2	10	0.30	0.0359	13,989	0.036	13,539
		5	0.30	0.0358	14,248	0.0359	13,790
		1	0.30	0.0365	13,489	0.0367	13,056
		0.1	0.30	0.0389	12,826	0.039	12,414
	7-8	10					
		5					
		1					
		0.1					
	7-10	10	0.30	0.043	11,632	0.0431	11,258
		5	0.29	0.0446	10,987	0.0447	10,634
		1	0.28	0.0443	10,105	0.0444	9,780
		0.1	0.26	0.0482	8,723	0.0484	8,443
	were not used in the construction of the stiffness master curve						

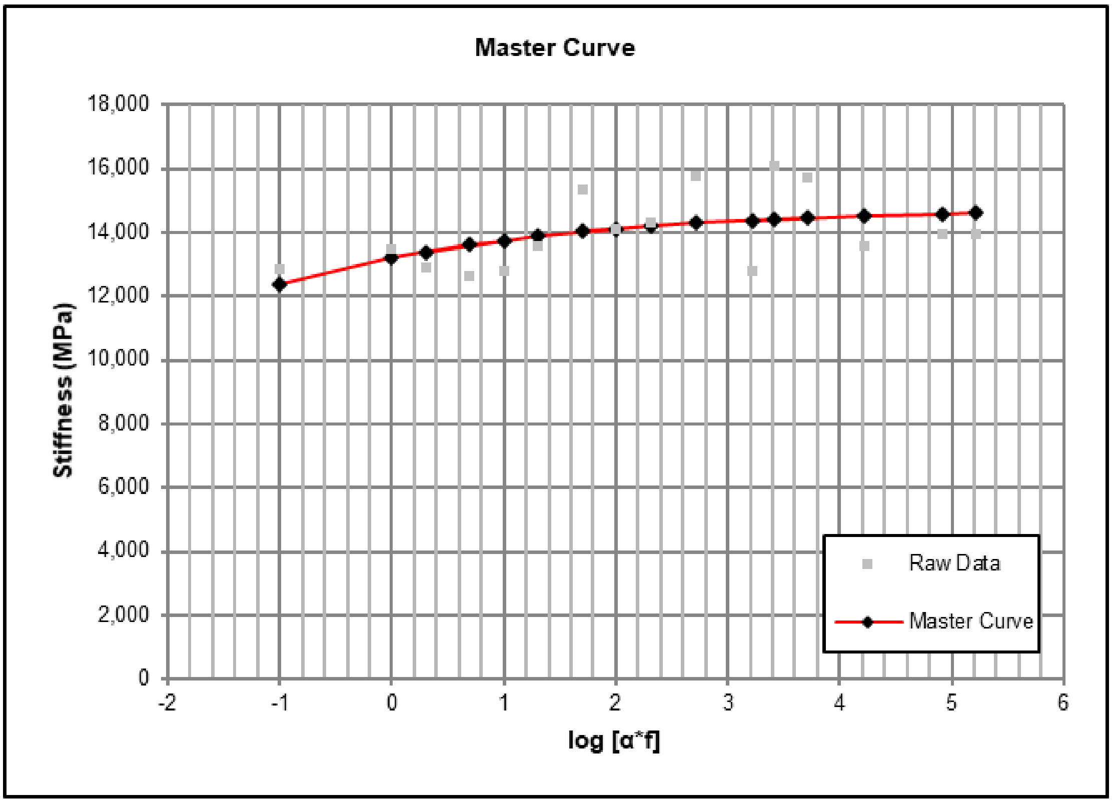

Figure E-9: Stiffness master curve of the F3.5C2.0 with asphaltic Poisson's ratio model

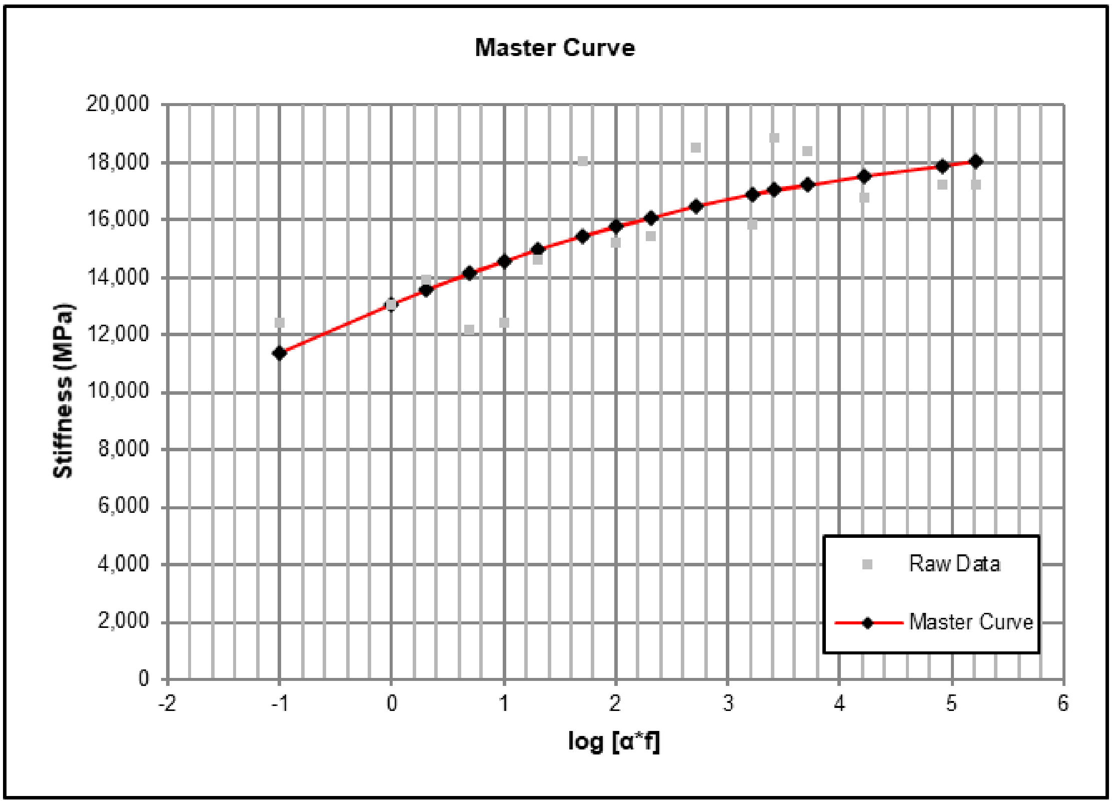

Figure E-10: Stiffness master curve of the F3.5C2.0 with constant (= 0.28) Poisson's ratio model

Table E-6: Stiffness test's results of F3.5C3.0 mix

F3.5C3.0 mix combination				As. Model Poisson's ratio		Constant Poisson's ratio	
Prüftemperatur (°C)	Specimen Nr.	Frequency (Hz)	Upperstress (MPa)	ε_{el} (‰)	Stiffness (Mpa)	ε_{el} (‰)	Stiffness (Mpa)
-10	9-1 (2)	10	0.57	0.042	19,509	0.0411	24,105
		5	0.57	0.0423	19,649	0.0414	24,278
		1	0.57	0.0428	18,852	0.0419	23,293
		0.1	0.57	0.0448	18,157	0.0438	22,435
	9-2 (2)	10	0.57	0.0426	19,175	0.0417	23,693
		5	0.57	0.0438	19,003	0.0428	23,479
		1	0.57	0.0435	18,521	0.0426	22,885
		0.1	0.57	0.0457	17,770	0.0447	21,956
	9-3 (2)	10	0.60	0.0412	20,923	0.0402	25,852
		5	0.60	0.0421	20,831	0.0412	25,738
		1	0.60	0.0418	20,391	0.0409	25,195
		0.1	0.60	0.0436	19,719	0.0426	24,365
0	9-7	10	0.51	0.0387	19,548	0.0381	22,926
		5	0.51	0.0394	19,623	0.0388	23,014
		1	0.51	0.0392	19,018	0.0386	22,305
		0.1	0.51	0.0411	18,327	0.0404	21,493
	9-9 (2)	10	0.54	0.0386	20,835	0.038	24,435
		5	0.54	0.0404	20,239	0.0397	23,737
		1	0.54	0.0393	20,165	0.0387	23,650
		0.1	0.54	0.0418	19,040	0.0411	22,330
	9-10	10	0.54	0.0401	19,989	0.0394	23,443
		5	0.54	0.0405	20,087	0.0398	23,558
		1	0.54	0.0412	19,151	0.0405	22,460
		0.1	0.54	0.0437	18,198	0.043	21,343
10	9-4	10	0.70	0.0463	24,574	0.046	26,512
		5	0.70	0.0572	20,275	0.0568	21,874
		1	0.70	0.0573	19,656	0.0568	21,206
		0.1	0.70	0.0578	19,606	0.0574	21,152
	9-6	10	0.62	0.0495	20,127	0.0491	21,714
		5	0.62	0.0498	20,450	0.0494	22,063
		1	0.62	0.0495	20,001	0.0491	21,578
		0.1	0.62	0.0509	18,894	0.0505	20,384
	9-8	10	0.62	0.0436	22,896	0.0433	24,702
		5	0.62	0.0435	23,493	0.0432	25,346
		1	0.62	0.041	24,203	0.0406	26,112
		0.1	0.62	0.0404	24,730	0.0401	26,680
20	9-1 (2)	10	0.52	0.0443	20,989	0.0445	20,315
		5	0.52	0.0458	20,618	0.0459	19,956
		1	0.52	0.0458	20,014	0.0459	19,371
		0.1	0.52	0.0479	19,245	0.048	18,627
	9-2 (2)	10	0.52	0.045	20,760	0.0451	20,093
		5	0.52	0.0472	20,085	0.0473	19,440
		1	0.52	0.0482	19,027	0.0483	18,416
		0.1	0.52	0.052	17,742	0.0522	17,172
	9-3 (2)	10	0.52	0.04	23,284	0.0401	22,536
		5	0.52	0.0409	23,097	0.0411	22,355
		1	0.52	0.0397	23,061	0.0398	22,320
		0.1	0.52	0.0418	22,034	0.0419	21,326
	were not used in the construction of the stiffness master curve						

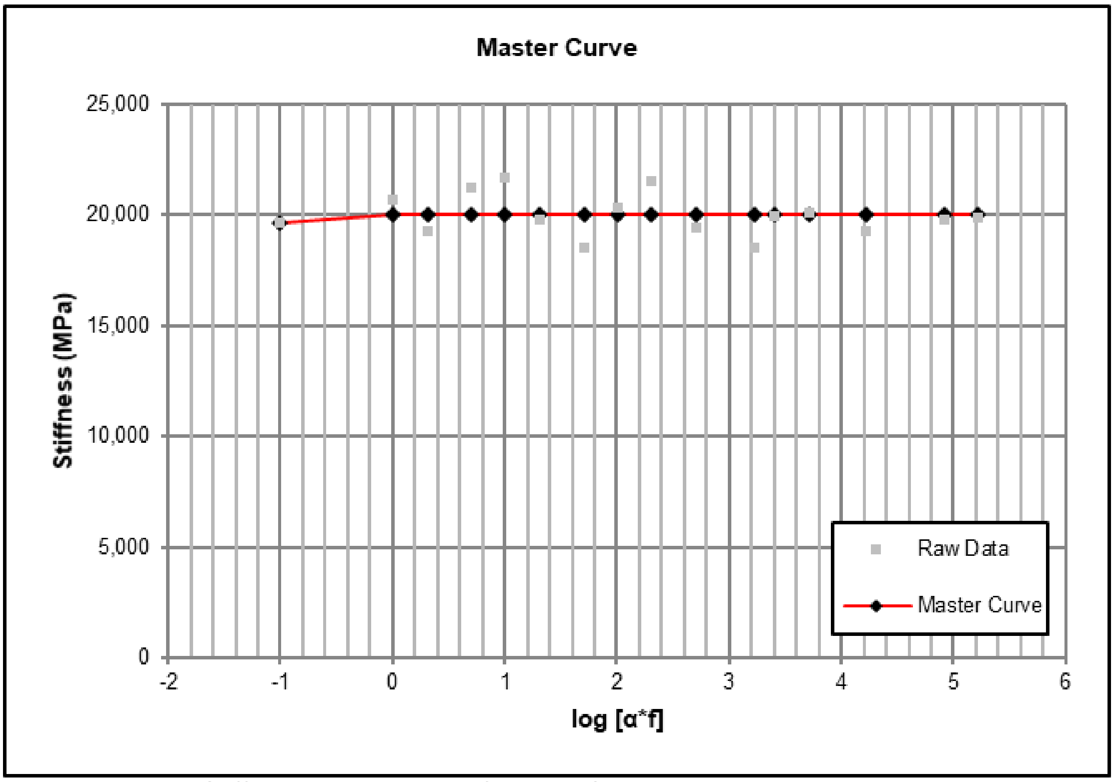

Figure E-11: Stiffness master curve of the F3.5C3.0 with asphaltic Poisson's ratio model

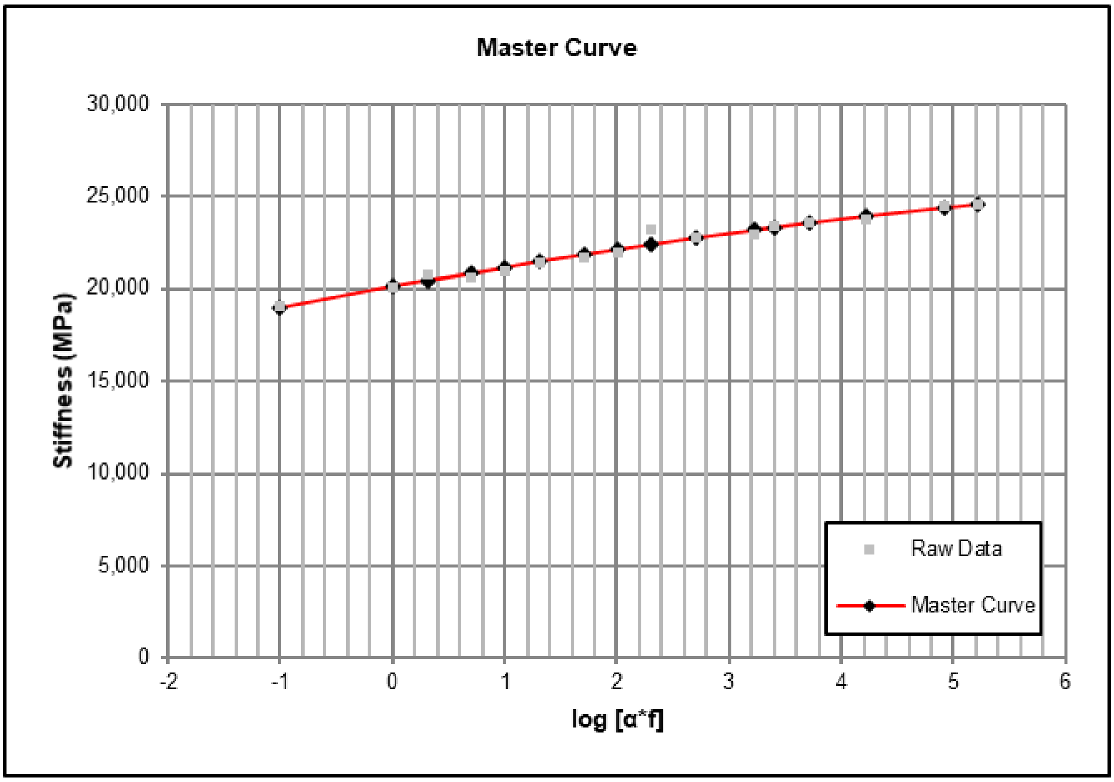

Figure E-12: Stiffness master curve of the F3.5C3.0 with constant (= 0.28) Poisson's ratio model

F- Stiffness-temperature equations of the materials used for pavement design example

As explained in section 5.4.1, the stiffness-temperature relation of the asphaltic materials and the FCSM (F3.5C1.0) were determined in polynomial 5 and used as the input for the design program. The below table shows the parameters of these polynomial equations.

Table F-1: The input data of the stiffness-temperature relation of the materials

Material	Polynomial 5 equation parameters					
	a_1	a_2	a_3	a_4	a_5	a_6
Wearing (DS)	-0.0000262	0.0000493	0.1958875	-1.1470295	-595.6933330	16,439.5167112
Binder (BS)	-0.0000037	-0.0010399	0.2487274	-5.5860201	-623.9286798	22,353.3559218
Base (TS)	0.0000036	0.0000759	0.0221677	2.8481974	-527.5490657	15,465.4614908
FCSM Base*	-0.0000002	0.0000384	0.0012165	-0.3869002	-40.528238	4,534.7461
* the F3.5C1.0 material at horizontal strain level of 0.1‰ calculated from general model						