

RILEM State-of-the-Art Reports

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Dariusz Sybilski *Editors*

Advances in Interlaboratory Testing and Evaluation of Bituminous Materials



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RILEM STATE-OF-THE-ART REPORTS

Volume IX

RILEM, The International Union of Laboratories and Experts in Construction Materials, Systems and Structures, founded in 1947, is a non-governmental scientific association whose goal is to contribute to progress in the construction sciences, techniques and industries, essentially by means of the communication it fosters between research and practice. RILEM's focus is on construction materials and their use in building and civil engineering structures, covering all phases of the building process from manufacture to use and recycling of materials. More information on RILEM and its previous publications can be found on www.RILEM.net.

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The information in this and similar reports is mostly pre-normative in the sense that it provides the underlying scientific fundamentals on which standards and codes of practice are based. Without such a solid scientific basis, construction practice will be less than efficient or economical.

It is RILEM's hope that this information will be of wide use to the scientific community.



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Advances in Interlaboratory Testing and Evaluation of Bituminous Materials

State-of-the-Art Report of the RILEM
Technical Committee 206-ATB

 Springer

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Preface

Bituminous materials and pavement technology currently undergo a dynamic innovative transition from a traditional empirical to a mechanistic way of engineering along with a change from phenomenological materials technology to materials science. This challenging transition process has been triggered by different factors, such as increasing performance requirements in terms of durability and bearing capacity, decreasing public tolerance to obstructions from repair and maintenance, tighter construction conditions through time and cost pressure as well as increasing environmental requirements for materials and construction. It is also driven by the understanding that asphalt roads can play an important role for future sustainable development, for example, by noise reduction, saving of material resources through recycling or use of marginal materials, and by saving energy during construction and production.

After gaining momentum by the end of the last century, this development has produced a great variety of new bituminous road materials and construction technologies worldwide which, as painfully learned by costly failures, can often not be handled sufficiently by traditional empirical materials engineering and testing, but are clearly asking for advanced test methods and improved understanding of mechanical behavior in a fundamental engineering and scientific way.

The *International Union for Testing and Research Laboratories for Materials and Structures* (RILEM) has therefore created over the years different Technical Committees working on specific material related questions regarding characterization and performance testing of bituminous binders, mixtures and pavement structures.

One of these Technical Committees was RILEM TC 206-ATB, on “Advanced Testing and Characterization of Bituminous Materials” which has been active between 2004 and 2010. It concentrated on simple and universally applicable performance based and physically sound fundamental test methods as well as homogeneous and uniform specimen preparation procedures for an ever increasing variety of bituminous binders and mixtures.

The RILEM TC 206-ATB (Chair: Manfred N. Partl, EMPA, Switzerland; Secretary: Emmanuel Chailleux, LCPC/*now IFSTTAR*/, France) was composed of five Task Groups:

- *TG 1 Binders* (Convener: Dariusz Sybilski, IBDIM Poland)
Evaluation of binder properties with respect to durability relevant distress accumulation, performance and application
- *TG 2 Mixture design and compaction* (Convener: Hussain U. Bahia, Univ. Wisconsin-Madison, USA)
Evaluation of laboratory compaction methods and models with respect to field compaction
- *TG 3 Mechanical testing of mixtures* (Convener: Hervé Di Benedetto, ENTPE, France)
Evaluation of existing test methods and models for different types of mixtures considering topics such as permanent deformation, micromechanics and size effects
- *TG 4 Pavement performance prediction evaluation* (Convener: Herald Piber, Bautechnik Carinthia, Austria, followed by Francesco Canestrari, Univ. Politecnica delle Marche, Italy)
Evaluation of test methods to assess structural behavior such as interlayer bond and investigation of sections for pavement performance prediction evaluation (PPPE)
- *TG 5 Recycling* (Convener: Chantal de la Roche, LCPC/*now IFSTTAR*/, France)
Evaluation of test and mix design methods for the use of materials with bituminous materials reclaimed from asphalt pavements, focusing on hot mix recycling.

This book presents the achievements of RILEM Technical Committee TC 206-ATB on “Advanced Testing and Characterization of Bituminous Materials” which were gathered over several years by intensive international interlaboratory testing and knowledge exchange between more than 50 members and experts from over 20 countries (number fluctuating over the years) as given in the member list below. In particular, it covers interlaboratory tests and experimental aspects of bituminous binder fatigue testing, also dealing with compaction and mixture design issues, by providing background on various compaction methods and imaging techniques for characterizing bituminous mixtures. This includes a comparative validation of a new 2D imaging software. In addition, specific experimental questions and analysis tools regarding mechanical wheel tracking tests are discussed, based on comparative results from different laboratories and by applying finite element techniques. Furthermore, a concluding analysis of a long term pavement performance prediction evaluation of rutting on a test section in Austria is given, followed by an extensive analysis and interlaboratory testing program on interlayer bond testing, incorporating three different test road sections which were constructed and monitored for this purpose. Finally, regarding hot mix recycling, the key issue of reclaimed asphalt manufacturing in the laboratory is studied and recommendations for laboratory ageing of bituminous mixtures are given.

This state-of-the-art book addresses to academics and researchers, intending to serve as a help for their experimental studies and for education of young researchers.

In this sense, it is expected to be a valuable source of information for PhD students with focused interest in bituminous materials and pavements. Furthermore, the book is expected to be acknowledged by experts in standardization committees as contribution and encouragement for improving national and international standards. It will certainly be also of interest for testing laboratories and test equipment producers for optimizing their equipment and methods. Finally, it will be a technical source for road authorities, professionals and practitioners who have to solve non-routine problems.

We hope that this book will be a valuable guideline and reference for further development of bituminous materials and asphalt pavement technology.

The editors
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The following individuals were active as members or as experts in the different working groups:

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- **TG4 Pavement Performance Prediction Evaluation** (Convener: Piber, H., followed by Canestrari, F.)
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Chapter 1

Introduction

Manfred N. Partl and Emmanuel Chailleux

Abstract A background on RILEM interlaboratory testing is presented with some comments regarding general development and objectives for advanced testing, providing also a general overview on requirements and needs. It is emphasized that development of today is driven by the evaluation of performance based material behavior, moving more and more from semi-empirical to fundamental test methods. Given this background, an updated version of the RILEM methodology for testing of bituminous pavement materials is presented with a short discussion of basic elements. This framework is the basis for this report on the achievements of RILEM technical committee TC 206-ATB on “Advanced Testing and Characterization of Bituminous Materials”. Furthermore, main principles and key elements for planning of interlaboratory tests are summarized containing points that proved also relevant for the interlaboratory tests presented and discussed in the following chapters of this book. This summary is intended as general checklist and practical tool for initiating and conducting future interlaboratory tests.

Keywords Bituminous materials • Basic concepts • General methodology • Interlaboratory testing guidelines • Planning checklist

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1.1 Background of RILEM Interlaboratory Tests

1.1.1 General

Evaluation and characterization of bituminous road materials in terms of performance related mechanical and chemo-physical properties are most important elements for generating progress and improved knowledge in application and development of bituminous road materials and systems on a global scale and in a sustainable way. In order to determine those properties, unified conclusive test methods have to be available, which are as simple as possible, efficient, refined and based on a sound physical background.

Such advanced test methods are needed as a basis for technical and economical global interaction since they allow comparing and defining the quality of materials independently of their origin and production. This is important for reducing trade barriers and for defining generally accepted state-of-the-art requirements by road administrations and road owners which also depend on a sound data basis for risk assessments and comparable life cycle considerations. In fact, unified test methods are the backbone for building up databases which are reliable and useful on a global scale. Advanced test methods are also essential for worldwide technical and scientific research interaction and knowledge exchange. They are necessary for the scientific understanding of behavior of the material and its structure from a multi-scale point of view, i.e. from small to big scale behavior. In addition, unified test methods represent also important pillars for theoretical modeling and design because they often deliver the input data and are therefore crucial for the accuracy of the theoretical predictions.

Interlaboratory testing, evaluation and characterization of bituminous materials based on their mechanical properties form a key issue to achieve the goal of unified test methods. These interlaboratory tests must be performed globally, or at least inter-continentially, in order to achieve most impact. Therefore, RILEM has devoted much of its effort in these pre-normative activities for bituminous materials during the last decades [1–10].

1.1.2 Advanced Testing of Bituminous Materials

Testing of bituminous materials has traditionally been driven by empirical and technological characterization of properties with major focus on quality assurance. Nowadays, it is generally agreed that this type of material characterization is too limited in its practical significance as input for engineering design and as tools for describing the increasing variety of bituminous materials. Hence, testing is now driven by the need of determining performance-based and performance-oriented properties that can be used not only for quality assurance but also as input for mechanistic-empirical pavement design methods. Focus in developing test methods

is clearly driven by overcoming the hierarchical levels of knowledge, i.e. to manage the shift from pragmatic (empiric) to technological (semi-empiric) to fundamental (scientific) testing and characterization methods.

The objective of advanced testing can be manifold. Search for more meaningful characterization and identification of production and manufacturing quality is one major goal. This is particularly important for standardization of products and systems. The second important goal is gathering fundamental information on the intrinsic mechanisms that determine the material behavior. The third main objective is the experimental validation of models that are based on fundamental material properties. These models should be able to describe the material behavior not only under idealized lab conditions but also in other more complex situations such as lab model systems tests, full scale field tests and accelerated pavement tests. The fourth major goal is the need for improved understandings of the material performance with respect to its practical use in terms of

- environmental impact, i.e. pollution, energy consumption and health,
- functionality, i.e. mechanical resistance (stiffness, fatigue, permanent deformation, etc.), road user expectations (safety, driving comfort), chemo-physical properties (water transport, noise reduction),
- sustainability, i.e. better use of material resources such as recycling of reclaimed asphalt pavements (RAP), re-use of secondary materials, marginal materials, substitute and regenerative materials.

These many objectives demonstrate that advanced testing is not a priori identical to the determination of mechanical properties. It also comprises determination of thermal properties, mechanisms of aging (e.g. influence of oxidation, heat, UV radiation, etc.) and healing, as well as determination of material behavior in contact with other substances such as water and chemicals (e.g. special additives and agents).

Testing of mechanical properties and behavior is still very important, of course. This is true for the interaction between the different components of bituminous materials in the undamaged elastic range and, in particular, for the behavior during evolution of damage and aging, considering time and temperature dependent viscous and plastic flow with hardening as well as failure and post failure mechanisms with softening. However, aging is not necessarily a negative phenomenon, resulting in deterioration of the material which is eventually slowed down by self-healing mechanisms or life-extending treatment (e.g. rejuvenators). In particular, with emerging new generations of energy saving bituminous pavement materials, aging may also have positive effects, resulting in maturing and improvement of properties (e.g. during carefully processed curing). Unfortunately, both negative and positive aging, are not well understood yet and require special advanced test methods for in depth investigations.

Development of advanced testing has made much progress in the last decades but has also suffered some setback which cooled down the optimism as it radiated from the US strategic highway research program in the 1990s of the last century. This is true, for example, with respect to fatigue testing, Superpave shear testing and the optimism in finding one simple performance test for asphalt pavements. On the other hand, new exciting developments such as the introduction of test

methods like X-Ray Computer Tomography (CT) as well as the increase of computational power for elaborating more sophisticated material models has contributed to the fact that the energy is still there.

A clear tendency can be observed towards a multi-scale approach, investigating and modeling the materials on a nano (nm... μ m), micro (μ m...mm), meso (mm...dm), magno (dm...dam) and mega (dam...km) scale. With bituminous materials, the nano scale is still widely unknown scientific territory, partly because of the experimental tools of today, which are of limited use for investigating these highly temperature and time dependent organic materials. For the magno and mega scale, on the other hand, the situation is different. Here, the level of knowledge is broader but in many cases only of limited general value because of the many influence factors that affect scientific testing on a 1:1 scale, such as construction parameters, climate, location, traffic characteristics and experimental costs. In fact, testing on magno and mega scale is often extremely expensive since constructing and operating test roads for destructive testing takes a lot of effort, expensive instrumentation and has to be done without disturbing the real traffic. Hence, for such tests, special linear, circular or elliptical test tracks and short road sections either in open air or in environmentally conditioned test halls have to be constructed and tortured with one of numerous stationary or mobile accelerated traffic simulators.

Generally, advanced test methods should be valid for many different types of bituminous materials, covering a broad application range in terms of temperature, strain, time etc. As far as asphalt pavement materials are concerned, they should ideally be able to cover all three basic concepts of the structural load carrying function of asphalt mixtures as shown in Fig. 1.1, i.e.

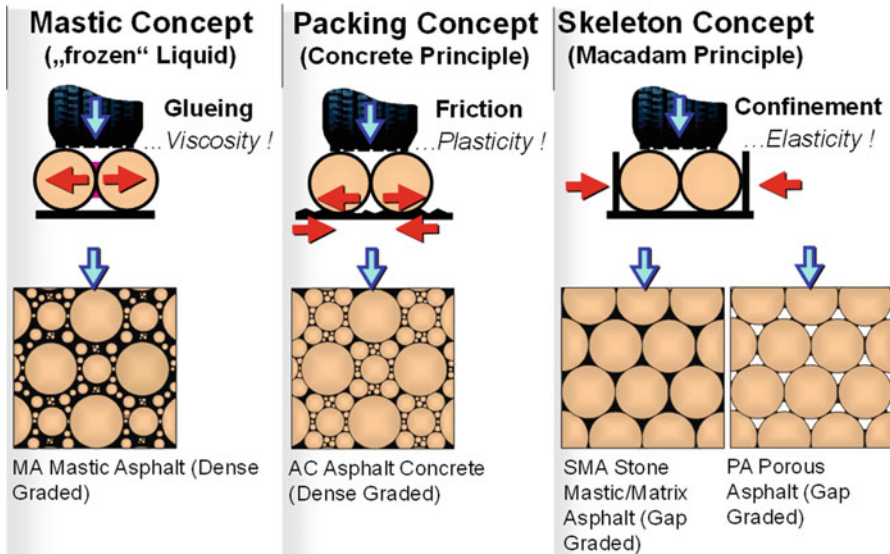


Fig. 1.1 Basic concepts for structural functioning of asphalt pavement mixtures

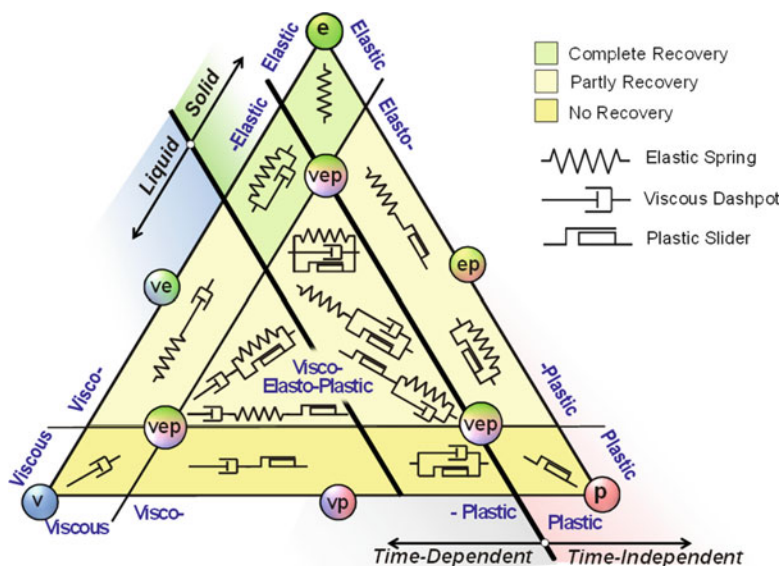


Fig. 1.2 Rheological triangle

- **mastic concept** (frozen liquid principle),
i.e. high binder content and virtually no air voids; aggregates “swim” in bituminous mortar matrix; load is mainly carried by the binder
- **packing concept** (concrete principle),
i.e. densest packing of aggregates with minimal binder film thickness; load carried by aggregates and binder
- **skeleton concept** (macadam principle),
i.e. corn-to-corn contact and interlock of stones; load primarily carried by aggregate skeleton; lateral support and confinement within layer necessary.

In any case, the range of validity of application should be clearly indicated for each test method in order to avoid applying tests for types of materials they were not designed or validated for.

In addition, advanced test methods should provide information on fundamental material properties that are universal and not biased by size effects and boundary conditions of the test setup. Hence, results should be useful directly for input in mechanical and chemo-physical models and calculations. Such fundamental material properties are necessary for a wide range of multi-scale modeling of temperature and moisture dependent elastic, viscous and plastic stress-strain behavior, as depicted schematically in Fig. 1.2, as well as for modeling of damage, healing and aging. It should be noted, that the elastic spring and viscous dashpot in Fig. 1.2 are not necessarily representing linear but also possible nonlinear behavior. In the same way, the plastic slider is not necessarily perfectly plastic. These rheological models for elastic, viscous and plastic behavior may be understood as qualitative help to define certain basic phenomena of stress-strain response, such as the difference

between solid and liquid or between time-dependent and time-independent behavior.

Advanced test methods should also allow clear quality assessment and unequivocal material identification (fingerprint). They are also required to be optimized in terms of measurement error. This means high precisions, i.e. providing results that are clustering closely together, as well as high accuracy, i.e. providing results that are arranging around the true value. Advanced test methods should not only be repeatable in the same lab with the same equipment, condition and personnel but also reproducible under similar conditions by other labs with other equipment and personnel. In order to reduce the risk of errors, testing should be easy to perform with minimum effort and training. Ideally, equipment should consist of simple and affordable equipment. A high degree of automation is certainly an advantage in reducing human errors, but careful check of the electronic system and validation of the software is essential, of course.

1.2 Methodology for Testing of Bituminous Pavement Materials – Basic Elements of a Testing Framework

Research performed by the RILEM technical committees on testing of bituminous materials follows a general methodology that has been presented in [5] but has been extended and refined in the following years as shown in Fig. 1.3.

The main elements of this methodology consist of four experimental fields where testing needs to be done (blue shaded areas) and two yellow shaded fields where pavement engineering is predominant. However, note that pavement engineering overlaps also most of the experimental fields as indicated by the purple shaded area. This is particularly true for identification and characterization, mix design and performance prediction as well as validation and performance tests. The different steps as shown in Fig. 1.3 can be described as follows.

The methodology starts with six preparatory steps:

1. *Fundamental scientific tests and identification.* This is basically a pre-standardization research activity that deals with acquiring experimental knowledge on new material components (binder, fibers, additives, pavements reinforcements, special aggregates, marginal materials, substitute or regenerative components, etc.) and their effect in different bituminous pavement mixtures. It includes also testing of asphalt mixtures produced in hot (146–250 °C), warm (100–145 °C), half-warm (50–95 °C) or cold technology (10–50 °C) containing reclaimed asphalt pavement material (RAP) or re-used secondary material components.
2. *Identification and characterization* of the base components (binder, additives and different aggregate fractions, reinforcement materials and other new components) and of the mixture composition (grading curve, proportions of the components including the binder). Identification and characterization

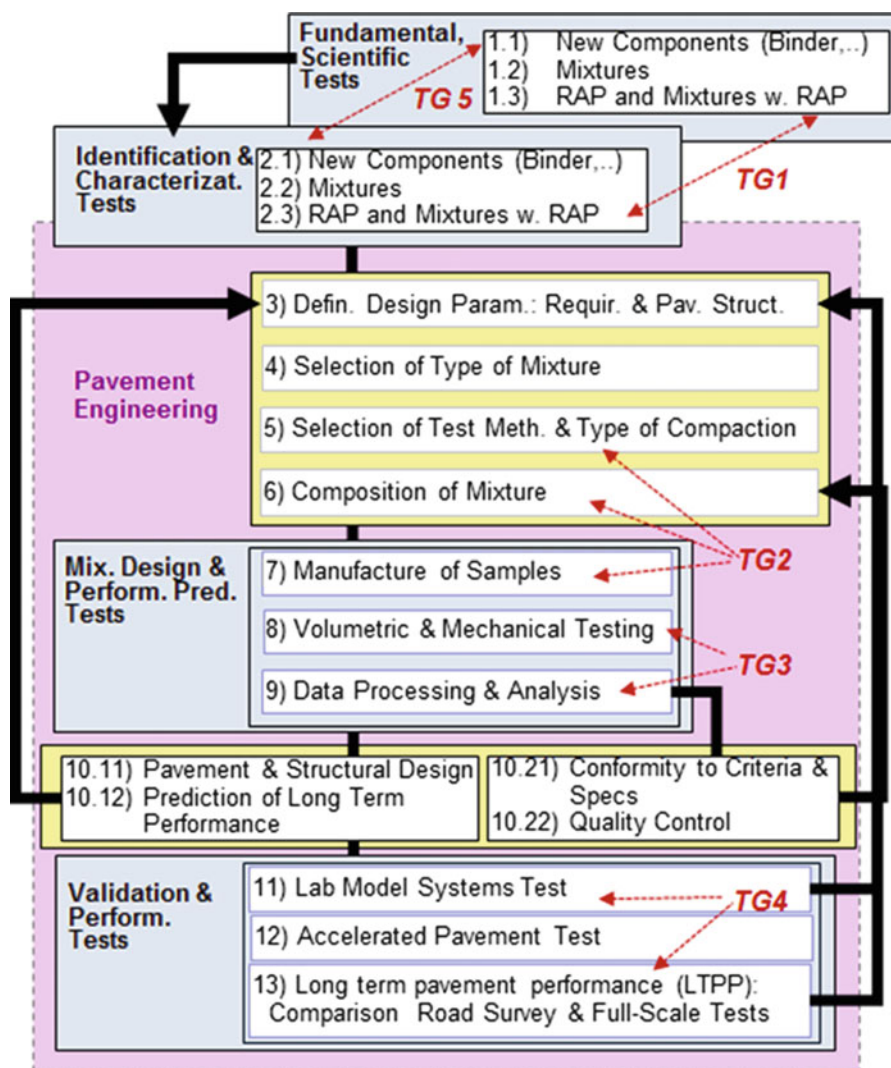


Fig. 1.3 Basic elements of a revised methodology for bituminous mixtures including the focus areas of the different Task groups of the TC 206-ATB (blue shaded areas are parts where testing has to be done, purple shaded area denotes field of pavement engineering)

testing is a type testing procedure, which has to be done for identification purposes (fingerprints) and is needed for quality control during production and for development of new products. Hence, it is not necessarily connected to specific in-field problems or requirements but it is the basis for a rough general screening of the technical possibilities during definition of the design parameters. To compare different products on a common basis, generally accepted test methods have to be used.

3. *Definition of design parameters* with respect to requirements (loading, climate, environmental aspects, life cycle, etc.) and pavement structure (including the position and function of the material in this structure) for a specific design job.
4. *Selection of type of mixture* such as asphalt concrete, stone mastic asphalt, open graded asphalt, overlays, etc. which is expected to have the best chance to meet the requirements formulated under step 3
5. *Selection of test methods as well as type and degree of compaction* suited to assess performance with respect to fatigue, permanent deformation, cracking, environmental requirements etc..
6. *Composition of mixtures* based either on experience, theoretical considerations or on the results from previous mixture design iterations.

The next three steps concentrate on mixture design and performance prediction testing and consist of

7. *Manufacture of samples*, i.e. mixing and compaction of the mixtures, final shaping and conditioning
8. *Volumetric and mechanical testing* including determination of sample composition (binder content, air voids) and testing of modulus, fatigue, permanent deformation, thermal cracking, etc..
9. *Data processing and analysis* with respect to
 - volumetric characteristics,
 - mechanical and chemo-physical characteristics,
 - environmental aspects,
 - statistical assessment.

Step 9 will lead to a set of data which can be used in one or both of the following two actions:

10.1 *Pavement design and modeling*

- Structural design
- Prediction of long term performance

10.2 *Conformity check*

- Conformity to criteria and specifications as basis for quality control
- Quality control

The use in design and prediction models (step 10.1) may finally lead to validation and performance tests, i.e.

11. *Lab model system test*
12. *Accelerated pavement testing (APT)* in the lab or in the field
13. *Long term pavement performance (LTPP)* tests in the field including
 - comparison with road survey results,
 - full-scale tests.

Steps 10 or 11–13 may lead to negative conclusions with respect to the material, and thus, the procedure needs to be repeated again with improvements concerning the selection of the base components and/or the mixture composition, eventually also affecting the selection of type of mixture and the test methods. It can also lead to a review of design parameters and pavement structure, e.g. with respect to loading or life cycle.

In the RILEM technical committee TC 206-ATB five task groups TG1 to TG5 were dealing with different subjects within this methodological framework, as indicated by dotted lines. The results of these activities are presented in the individual Chapters of this report.

1.3 Principles for Planning Interlaboratory Tests

1.3.1 *General*

There are already numerous documents available on how to conduct interlaboratory tests in an efficient and conclusive way. One example is the Standard ISO/IEC 17043:2010 “conformity assessment-general requirements for proficiency testing” that replaces the old ISO Guide 43 “Development and operation of laboratory proficiency testing”. It contains valuable and quite detailed information in that respect, specifying general requirements for the development and operation as well as for the competence of providers of proficiency testing schemes. Proficiency testing means the evaluation of participant performance against pre-established criteria by means of interlaboratory comparisons.

However, the ISO/IEC standard focuses primarily on the interlaboratory comparison and performance evaluation of different participants based on standard procedures and criteria. This objective is different in many ways from interlaboratory tests performed by RILEM where, instead of the performance of the different participants, the scientific, methodical search for the most accurate way of determining fundamental, physical, performance oriented material and systems properties is predominant. Hence, RILEM interlaboratory tests have a strong pre-standardization path-finder character. There is no question that the principles formulated in the ISO/IEC 17042:2010 document are also true in case of RILEM interlaboratory tests. However, since the objectives are different, strict application of the ISO/IEC standard is not always possible.

Nevertheless, it is very clear that the success of RILEM interlaboratory tests also depends on the seven key elements that have to be taken into account in a very early planning phase:

1. the aim as well as the answers and outcome that are expected
2. the systematic and methodology to be chosen
3. the selection of test methods, hypothesis and materials
4. the number and quality of participants
5. the choice and methodology of both analysis and modeling

6. the way of knowledge transfer, i.e. information and implementation regarding the outcome and answers
7. the organization as well as way and means of realization

Generally, in addition to these mostly technical key elements, ownership and other legal matters should also be defined in a very early stage. However, in case of interlaboratory tests performed by an organization, this issue may already be covered by the regulations and statutes inside the organization.

1.3.2 Checklist of Major Points

In this paragraph, a short checklist for planning and performing interlaboratory tests is given. It may help as general practical tool also containing points that proved relevant for the interlaboratory tests presented and discussed in the following chapters. Once the seven key elements of the planning phase are determined and defined, the following specific points should be further clarified:

1. Is the scope and aim clearly defined in terms of methods, materials, instruments, data to be determined or evaluated?
2. How and on whose shoulders are responsibilities distributed?

It must be clear who is leading and managing the interlaboratory test in a technical and administrative way and who is also active member in the organizing team. An organization chart may define the different bodies that have assigned tasks, such as scientific steering panel, evaluation task force, communication and reporting body etc.. Responsibilities of the different bodies in the organizing team should also be defined. It is also important to clarify the role of the test participants within this organizational framework.

3. Who are the participants?

This question not only needs to be answered in terms of participating persons and institutions, but also in terms of responsible people. It should also be clarified who would be responsible in case of a change of personnel.

4. What are the competences of the participants?

It must be verified if the participants have sufficient knowhow, skills and resources to participate. This includes checking if the personnel has sufficient expertise and training for the work. In addition, one has also to explore what priority and effort the participants are willing to attribute to accomplish their work.

5. Do the participants have the necessary test equipment and devices and are all necessary tools available for data evaluation and data analysis?

In this context the following points should be clarified:

- Which devices and tools are used?
- Is it ensured that devices and tools are working properly in terms of calibration, maintenance, control procedures and trained operators?

- Is the testing, data acquisition and evaluation software validated?
- Are the devices compatible with the testing guidelines?
- Do the electronic tool for data and document interchange comply with the communication guidelines?

6. Are confidentiality issues solved?

Full success is only achieved if the participants are willing to share their knowledge, expertise, results and findings in an open collaborative scientific way. There may be restrictions of confidentiality, of course, but these restrictions have to be clarified and written down in advance. In order to avoid problems by mentioning brand names it may be more informative and of general value to describe the material through its chemical composition or, if not possible, through neutral labeling.

7. Are guidelines, clear descriptions or standards available that define precisely the testing, evaluation and modeling procedures?

Depending on the goal of the interlaboratory test, this does not mean that everybody follows the same procedures, but each procedure considered in the test must clearly be described. Hence, the description of the procedures must be unambiguous, practicable and clear enough in order to make sure that the results are independent of the lab and personnel. This is a must in cases where repeatability and reproducibility of a procedure are of major focus.

Unfortunately, standards are often written in way that is not based on purely technical knowledge, but on compromises of interests and the “lowest common denominator” principle. Experience shows that this often keeps too many open options, raising too many questions that often result in misinterpretations and disputes. This does not mean, of course, that descriptions of procedures should suffer from an overkill of rules, but it clearly means that decisive technical points must be precisely defined.

In terms of guidelines and documents for an interlaboratory test, the following additional questions should be answered:

- Do the guidelines, instructions and documents comply with the goals?
- Are instructions available on how to:
 - Perform sampling?
 - Handle the specimens, in terms of transport, marking and storage?
 - Prepare the specimens for shipping and testing (including conditioning in the lab before testing)?
 - Perform the testing?
 - Communicate and interchange data and documents between the participants and the organizing team in a compatible way?
- Are all test procedures clearly described?
- Are guidelines available for data acquisition (forms, what data should be acquired) and data evaluation?
- Do guidelines exist for recording, taking notes and reporting?
- Do all participants have all necessary guidelines and documents?
- Do all participants understand the guidelines and documents?

8. How and by whom is sampling of the test specimens and materials done?

This implies the check of the following questions:

- Are the people who perform that sampling trained and instructed for this job?
- If sampling is done by third party, is a representative of the participants or the organizing team present?
- Is it ensured that all necessary information are recorded, such as location, orientation, climate, date, weather and other influencing factors during sampling, such as tools, personnel, quantity?
- Are the samples correctly and unambiguously labeled?
- Are the handling instructions regarding specimen transport and storage considered?
- Is the labeled material clearly described in terms of composition, recipe, batch, dimensions, weight, age, origin, condition etc.?
- Is it ensured that all necessary information regarding storage and the procedures for specimen preparation for each specimen are clearly documented?

9. Are retained samples set aside?

This is an important point for different reasons. Firstly, particularly during in path-finder interlaboratory tests, new knowledge may be produced and new questions may arise which would need new specimens to be investigated; secondly new participants may join the group as the investigation goes, since interest may grow in particular when the investigation last several months; thirdly, retained specimens may be very valuable to achieve synergies with other interlaboratory tests where somewhat different questions are studied.

10. How and according to what criteria are the specimens and materials distributed to the different participants?

This includes also the question on the kind of packaging for shipping and the question of who takes care of the shipping costs.

In particular with bituminous materials, good expertise in packing is required due to their high time- temperature dependency. Specimens should always be packed such that no deformation of the specimen, no excessive stresses, no moistening and no interaction with the packing material during transport are created. In addition, special effort in individual labeling is needed, because some markers may lose readability over time and under transport conditions.

11. What is the time-table to perform testing, analysis and modeling?

Fixing the time-table is particularly important in cases, where aging and conditioning of the specimens may play a role.

12. Who does the data evaluation and draws conclusions?

This has to do with the question on how the data are presented, handled and interpreted. It deals with what evaluation methodology, statistical models and computer software are used. It also implies to ensure that skilled experts with good statistical and technical knowledge are doing the job and that these experts are supported by the participants in case of giving additional back ground input.

13. Is the intellectual ownership of the data and documents clarified?

In case that one organization organizes the interlaboratory tests generally clear rules on this subject may already exist. However, these rules have to be communicated to everybody involved or potentially involved in the tests. In particular this raises the following questions:

- Who owns the data and where are the data stored?
- Who has access to the data base and under what conditions?
- How are the data maintained?

14. How are the results and outcome communicated?

Information and implementation regarding the outcome and answers of the interlaboratory test, in particular regarding the aim and lessons learned, is the most important issue. In this context the following points need consideration:

- Who writes the final report and who takes authorship?

Some organizations have rules regarding the authorship, e.g. only members of the organization can be listed as authors. These points must be clarified and communicated in a very early stage to the participants.

- In what form are the final report and documents published?
- How and what kind is the feedback given to participants?

15. Is the content of the final report and documents clear and properly reviewed such that it can be published in the name of the organizing team of the interlaboratory test?

This question implies the following points:

- Is it ensured that all documents from the participants including all information on sampling, transport, preparation, testing is available?
- Were all the guidelines considered accordingly?
- Does the report consider the confidentiality issues?
- Does the interpretation and statistical analysis of the data in the report comply with the state of the art; for example, does it comply with the standards ISO 5725 on accuracy (trueness and precision) of measurement methods and results?

16. Are the questions of the costs clarified?

17. Do the participants agree and accept the procedure of the interlaboratory test?

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Chapter 2

Binder Testing

**Dariusz Sybilski, Hilde Soenen, Marcin Gajewski, Emmanuel Chailleux,
and Wojciech Bankowski**

Abstract In this chapter, a state of the art on the basics of the fatigue phenomenon of engineering materials is presented, with special attention to asphalt materials and bituminous binders. Since engineering materials are exposed to complex mechanical and environmental loadings (e.g. temperature and humidity variation etc.) asphalt mixture fatigue tests have been developed and widely used in the last decades. These tests are time consuming and relatively expensive. Hence, binder fatigue tests might be an effective preliminary evaluation of material helpful for mixture fatigue life testing.

Binder fatigue tests are used to investigate the fatigue behavior of bituminous binders and mastics, to compare different binder types, and to investigate the role of the binder in mixture fatigue behavior. The most common equipment for binder fatigue tests has been the plate-plate Dynamic Shear Rheometer (DSR). Other test geometries have been proposed as well. Even if DSR testing has become popular, there are up to now little data available on the reproducibility of this test. Task Group 1 (Binders) of the RILEM TC 206-ATB (Advanced Testing of Bituminous materials) undertook the task of organization of the RILEM Round Robin Binder Fatigue Test (RRRBFT). The purpose of this interlaboratory test was to investigate the repeatability as well as the reproducibility of binder fatigue tests.

Keywords Bituminous binders • Fatigue life • Laboratory testing • Test procedure • Dynamic Shear Rheometer • Binder - asphalt mixture comparison

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2.1 Bituminous Binder Fatigue Testing: State-Of-The-Art

2.1.1 *Fatigue Phenomenon*

2.1.1.1 Basics

In engineering materials exposed to complex mechanical and environmental loading (e.g. temperature and humidity variation etc.) constant micro-structural (or structural depending on material types) changes occur, causing a drop of strength. In general such effects are complex, but at macroscopic scale can be seen as so called material damage.

Depending on material type, the damage phenomena can be understood differently, e.g. in polymers damage occurs by breakage of bonds between molecules chains, in case of fibre composites it happens by the separation of fibres from composite matrix, for concrete material damage begins by debonding between aggregates and cement, i.e. from micro-cracks, in the case of wood, damage is a result of large deformations of cellulosic cells, etc.

In frame of continuum mechanics it is possible to distinguish some groups of constitutive relationships, e.g.: elasticity, visco-elasticity, plasticity. Construction materials require a combination of such relationships with an estimation of elastic or plastic material properties ranges. Apparently material fatigue (depending on material type) can occur in each range. For elastic materials it happens at atomic level while for plastic materials it happens when lines of dislocations are moved producing plastic strains. The conclusion is that these two damage mechanisms have different scales but can be treated similarly on macroscopic level via proper determination of a representative volume element (e.g. Lemaitre [28]).

In damage mechanics (e.g. Kachanov [26]) there is the possibility to distinguish some characteristic types of damage. The most important types are presented below.

Brittle damage. Brittle damage occurs when a crack is initiated without a large amount of plastic strains, i.e. the ratio of plastic strain to elastic strain is below unity.

Ductile damage. Ductile damage happens simultaneously with plastic deformations larger than a certain threshold on the graph of stress (stress norm) as a function of strain. It results from the nucleation of cavities due to debonding between inclusions and the matrix which causes local plastic instability.

Creep damage. When materials have viscous properties, damage can occur at a constant stress level. Total strains gradually increase and cause irregular inter-granular cracks.

Low cycle fatigue damage. When materials are subjected to cyclic loading with large stress or strain amplitude values, damage develops with plastic deformation in three phases: incubation, nucleation and micro-crack propagation. In case of low cycle fatigue, the damage can be either inter-granular or trans-granular.

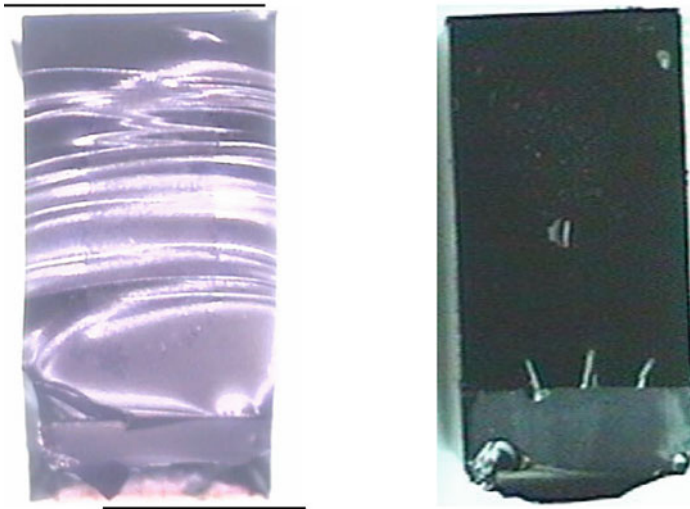


Fig. 2.1 Ductile and brittle fracture examples (Chailleux [15])

High cycle fatigue. This case is contrary to low cycle fatigue damage. Here, damage is observed for a higher number of cycles with lower amplitude of stresses or strains. Depending on material type subjected to such cyclic loading, the plastic strain at the meso-level remains small and is often negligible. Then damage symptoms can be observed at the micro-scale.

2.1.1.2 Fatigue in Asphalt Binder

In case of modified and unmodified asphalt binders it is worth noting, that depending on temperature, material can be treated as elastic or visco-elastic. For higher temperatures it is possible to assume that binder is a fluid with completely viscous properties. In this connection, for a wide range of temperatures, damage mechanisms observed in laboratory tests can differ from brittle cracking through ductile cracking up to high cycle fatigue damage (Figs. 2.1, 2.2 and 2.3). In the case of pavements which generally carry loads that are considerably lower (in normal functional state) than the load capacity, but in a cyclically repeatable way, one can assume that high cycle fatigue can occur.

2.1.1.3 Asphalt Pavement Cracking

Introduction

The most basic asphalt pavement deteriorations are permanent deformation and cracking. This order reflects also the ranking of deteriorations in the majority of world's regions (excluding cold climate regions).

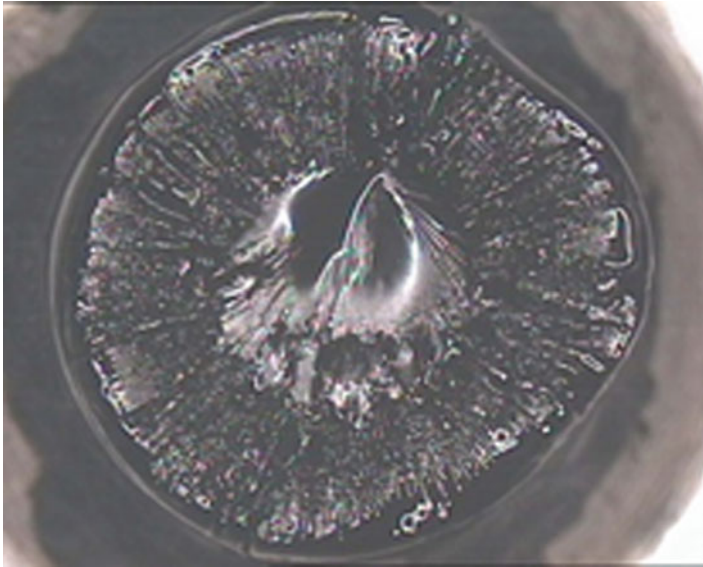


Fig. 2.2 Damage of asphalt binder sample(Binder A) subjected to DSR fatigue test at 10°C (Chailleux [15])

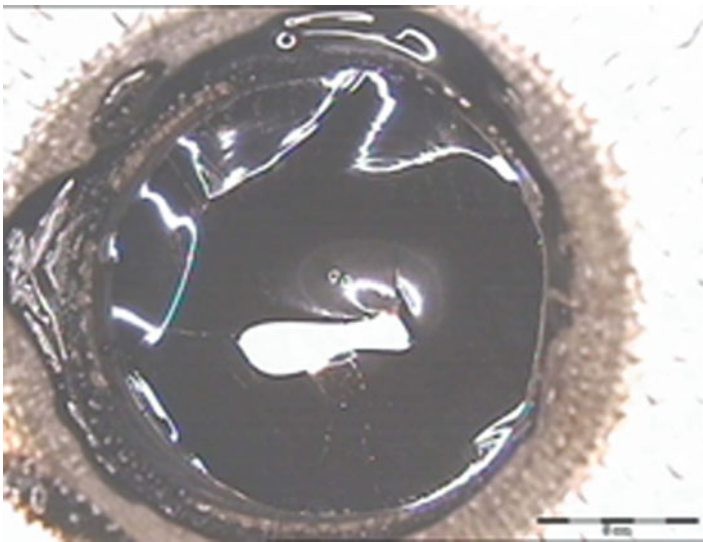
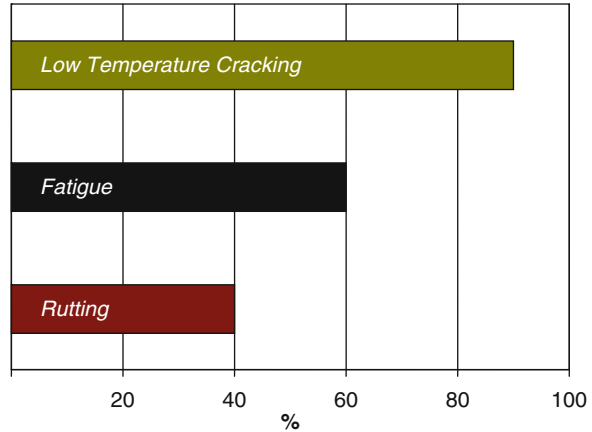


Fig. 2.3 Damage of asphalt binder sample (Binder C) subjected to DSR fatigue test at 10°C (Chailleux [15])

Fig. 2.4 Influence of binder on asphalt distress



Bituminous binder plays important role in the behaviour of asphalt and pavement structures. Its influence on various deterioration modes is shown in Fig. 2.4.

Cracking in asphalt pavement has various origins (cf. Vanelstraete and Francken [50], Jacobs [25]). Low-temperature and fatigue cracking presented in Fig. 2.4 are among the most frequently observed origins. Fatigue cracking may be generated by traffic load or temperature variation. Types of fatigue cracking may be:

- Top – bottom, with initiation at the top surface and propagation to the bottom,
- Bottom – top, with initiation at the bottom and propagation to the top.

Bituminous binder plays an important role in asphalt pavement fatigue phenomena. The majority of cracks in bitumen-aggregate composites run typically through the bitumen film and not through the aggregate grains (cf. Harvey and Cebon [23]). Cracks running through aggregates may be observed only at low temperatures. For fatigue cracking analysis and pavement fatigue life prediction, a medium temperature range from 15 to 30°C is most suitable (Deacon et al. [17]).

Theoretical Basis

For fracture mechanics, the crack nucleation and the development of the crack is a key issue (cf. Lemaitre [28]). In general, crack development may be described by an equation of the following type:

$$\frac{dc}{dN} = f(s, c, M_c, G_p, A_r, H) \quad (2.1)$$

where:

- s amplitude of stress for cyclic loading,
- c actual crack length,

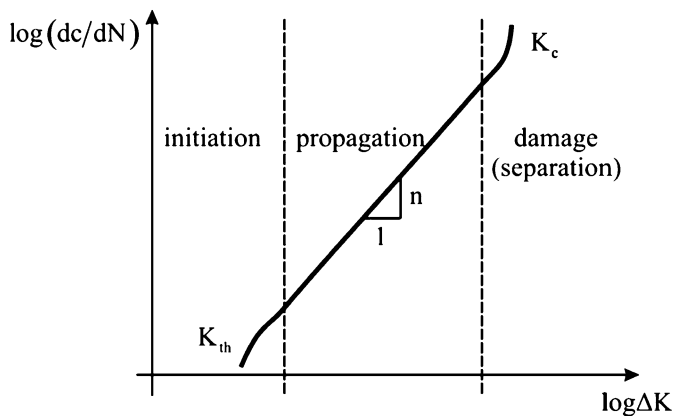


Fig. 2.5 Theoretical curve of increasing fatigue crack as a function of stress intensity factor

M_c stands for the material parameters,
 G_p describes crack geometry,
 A_r characterizes asymmetry of stress cycle,
 H describes the material loading history.

Of course, it is impossible to determine such a general function. Hence, some simplifications are needed. In this field, a lot of research was made by Paris and Erdogan [32]. They studied the characteristic behavior (cf. Fig. 2.3) of many materials in the crack propagation phase and proposed simple functions of the form $f(\Delta K, M_c, A_r)$, where $\Delta K = K_{max} - K_{min}$ is the difference between maximal and minimal stress intensity factor.

According to Fig. 2.5, it is possible to distinguish three basic phases: initiation, propagation and damage (separation). Value K_{th} corresponds to the limiting stress intensity factor, below which no crack initiation is possible. Parameter K_c is a critical stress intensity factor which characterizes the material's cracking resistance.

After initiation, crack propagation may be ruled by the Paris-Erdogan law

$$\frac{dc}{dN} = A \cdot \Delta K^n \quad (2.2)$$

as shown in Fig. 2.5. In Eq. (2.2) the notations are

ΔK difference between maximal and minimal stress intensity factor
 $\Delta K = K_{max} - K_{min}$
 N number of loading cycles
 C crack length, mm
 A, n parameters obtained from tests.

The number of loading cycles needed for crack propagation through the whole layer thickness h is specified by

$$N_f = \int_0^h \frac{dc}{A \cdot (\Delta K(c))^n} \quad (2.3)$$

The stress intensity factor K , describing stress values around a crack tip, plays an important role in fracture mechanics. It is a function of crack length, specimen geometry and loading conditions provided that the material is isotropic and homogeneous.

2.1.1.4 Fatigue and Healing Phenomena

Fatigue

Material fatigue is one of the commonly observed phenomena when material is exposed to multiple, cyclic loads. In particular, this phenomenon occurs also with asphalt or concrete road pavements cyclically loaded by vehicles.

Fatigue cracking, next to permanent strain accumulation, is the main type of asphalt pavement deterioration. These two deterioration modes are completely different but related in the sense that depending on the mix design an increase of asphalt pavement rutting resistance may lead to decrease of fatigue life and vice versa.

In pavement construction design, the fatigue criteria are employed as one of the main criteria for bound layers (asphalt, concrete, stabilized with hydraulic binder). Material fatigue, based on initiation and crack propagation (micro- and macro-scale), leads to a decrease of stiffness modulus. In classical fatigue analysis of asphalt, failure is defined as a point (number of cycles) when the value of stiffness modulus is equal to 50% of its initial value.

A single loading causes a material fatigue damage fraction D_i . According to the Palmgreen-Miner rule, the sum of fatigue damage fractions leads to the fatigue damage D as follows:

$$D = \sum_{i=1}^k \frac{n_i}{N_i} \quad (2.4)$$

n_i number of applied load cycles at constant stress level S_i ,

N_i number of loadings to the failure at constant stress level S_i ,

K number of loadings (number of intervals with constant amplitude),

D fatigue damage: $D \leq 1$ ($D = 1$ indicates failure).

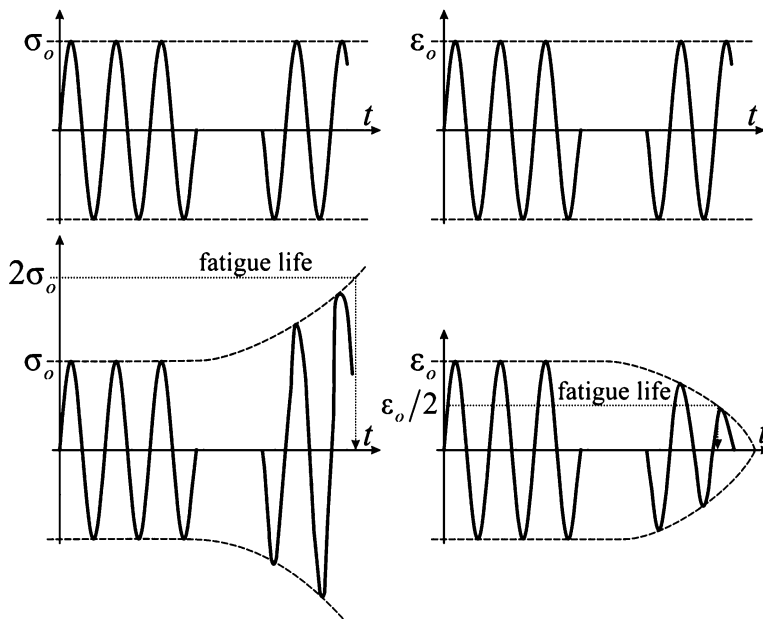


Fig. 2.6 Comparison of controlled strain and stress modes (Lundström [29])

Equation (2.4) is correct only under the assumption of linear accumulation of damage due to different levels of stress. Experiments show that Eq. (2.4) is valid if the loading sequences do not vary too much in amplitude but may give wrong results if the amplitudes differ by more than 10–20%, cf. Lemaître [28].

Healing

Healing is the process of micro- or macro-damage recovery in a material during the rest period of loading. The asphalt pavement healing phenomenon has been noted for years, but its relation to asphalt fatigue behavior is still not definitely clarified and explained. A lot of research is now devoted to these phenomena (cf. Lu et al. [30]; Hammoum et al. [22]; Bodin et al. [12]). Thanks to dynamic shear rheometers the healing phenomena can be studied in more detail (Lu et al. [30]).

2.1.1.5 Fatigue Life

Fatigue life of pavement constructions is defined as the number of standard axles passings until mechanical failure (i.e. cracking on the pavement surface). In laboratory tests, fatigue life is defined as a number of stress or strain cycles to failure of a specimen predicted by fatigue criteria (Fig. 2.6).

The fatigue law for asphalt is often expressed by:

$$N_f = K_1 \left(\frac{1}{\varepsilon_t} \right)^{k_2} \quad (2.5)$$

where:

- N_f number of loading cycles to fatigue failure,
- ε_t tensile strain at the critical point,
- K_1, k_2 calibration coefficients taken from laboratory tests.

In some construction design methods this general equation is transformed to the form where fatigue life is a direct function of asphalt stiffness modulus:

$$N_f = k_1 \left(\frac{1}{E^*} \right)^{k_3} \left(\frac{1}{\varepsilon_t} \right)^{k_2} \quad (2.6)$$

The relationship between parameters from Eqs. (2.5) and (2.6) is

$$K_1 = k_1 (E^*)^{-k_3} \quad (2.7)$$

where:

- E^* material stiffness modulus (norm of complex stiffness modulus),
- k_1, k_3 calibration coefficients obtained from laboratory tests.

Studies on fatigue of asphalt-based materials revealed healing phenomena in the material.

Healing of material is a partial restoration of initial properties (lost as a result of cyclic loading) at rest, during unloading before the next load application. Asphalt materials show an ability to heal thanks to thixotropic effects, i.e. the ability to restore initial viscosity after load removal.

2.1.1.6 Fatigue Life Criteria

Conventional Criteria

At present, commonly used and acknowledged conventional fatigue criteria of asphalt mixtures, define failure of a material as a situation when its stiffness modulus decreases to 50% of its initial value. In particular, fatigue life is considered as the minimum strain amplitude ε_6 , for which an asphalt sample can be exposed to one million load cycles.

Conventional fatigue criteria defining fatigue life are arbitrary criteria which consider only stiffness modulus instead of the overall material properties. It is also worth noting that for higher temperatures and loads, a decrease of stiffness modulus at the beginning of the test is significant and not always exclusively connected to the fatigue phenomenon. In this early phase the internal heating phenomenon dominates (Bodin et al. [12]; Bańkowski [11]).

A Dissipated Energy Approach Criteria

A new definition of fatigue life (denoted as N_1), based on dissipated energy originally formulated by Van Dijk [44], was proposed by Hopman et al. [24]. Most of the energy dissipated during fatigue tests is transformed into thermal energy. Nevertheless, the temperature growth is not significant, and cannot be interpreted as only reason for reducing the value of stiffness modulus (Pronk [37]; Pronk et al. [43]). A retained part of energy is related to mechanical fatigue of the specimen and can therefore be interpreted as dissipated energy. During a fatigue test, it is possible to observe a sudden change of dissipated energy which is interpreted by Pronk (Hopman et al. [24]) as the starting point of the last test phase, i.e. sudden specimen degradation.

In the fatigue process two basic stages are distinguished: initiation of micro-cracks (stage 1) and propagation of micro- and macro-cracks leading to specimen and material failure (stage 2). In both stages some energy is dissipated during a single load cycle:

$$W_n = \pi \cdot \sigma_n \cdot \varepsilon_n \cdot \sin \theta_n \quad (2.8)$$

where:

W_n dissipated energy, J/m³,
 σ_n stress amplitude, MPa
 ε_n strain amplitude, $\mu\text{mm/mm}$
 θ_n phase shift angle, deg.

This phenomenon can be demonstrated in a graph showing dissipated energy ratio (DER) as a function of the number of load cycles. The dissipated energy ratio can be expressed by equation:

$$W_n = \frac{n \cdot w_0}{w_n} \quad (2.9)$$

where:

n number of loading cycle,
 w_0 energy dissipated in first cycle,
 w_n energy dissipated in n-th cycle.

Fig. 2.7 Dissipated energy ratio in controlled strain mode (Pronk [38])

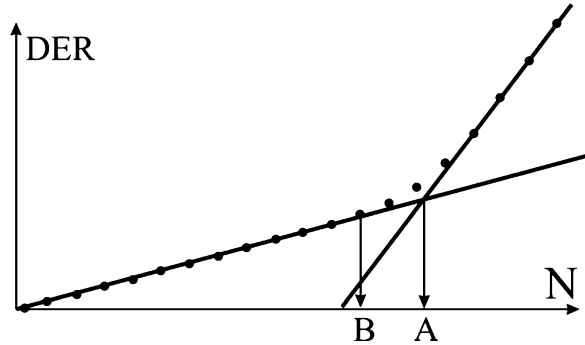
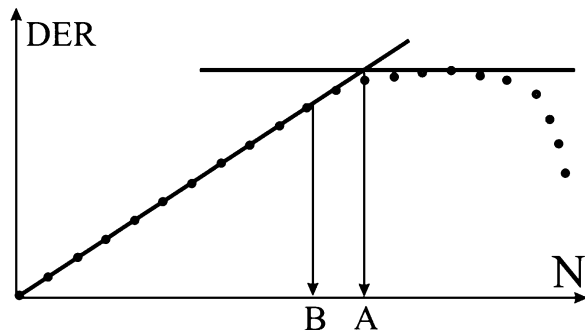


Fig. 2.8 Dissipated energy ratio in controlled stress mode (Pronk [39])



Using the dissipated energy fatigue criteria, the new fatigue life definition noted as N_1 was developed. N_1 describes the number of cycles, when the graph of dissipated energy ratio as a function of number of cycles is no longer linear (Pronk [38]). Such a graph has a different character in strain or stress controlled mode (Figs. 2.7 and 2.8).

However, it turned out that practical application of this criterion is difficult to apply, because of ambiguity in the point where the graph of dissipated energy ratio is no longer linear. Hence, two graphical methods were proposed. The tangent intersection method (point A) leads to an overestimation of fatigue life. The second method relies on finding a point on the graph, where the function changes from linear into a curve (point B). Such results may be subjective, depending on the person doing the calculations, the density of points obtained from the laboratory test, the scale of the graph, etc. (Pronk [41]). In such a situation it is necessary to develop a method which objectively and precisely allows determination of the fatigue life N_1 .

Criteria established this way are based on observations of the fatigue process, contrary to conventional criteria, which defines arbitrary fatigue life as a number of cycles needed to lower the elasticity modulus to 50% of its original value. Pronk preliminary laboratory tests with four point beam bending showed, that it is possible to compare results from strain or stress controlled tests even for different frequencies (Pronk and Hopman [42]). Then, it was postulated that a new definition

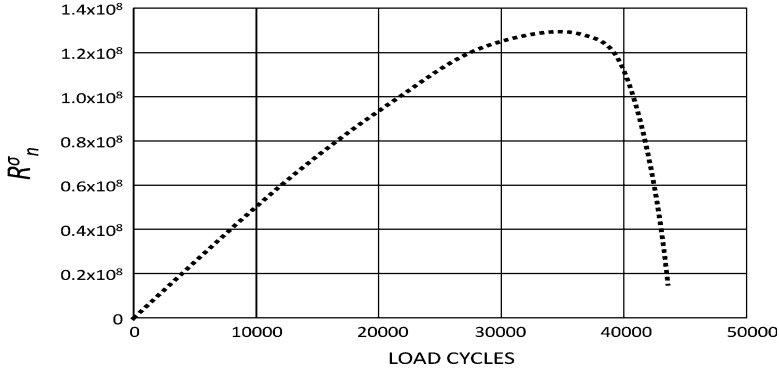


Fig. 2.9 Reduced dissipated energy ratio R_n^σ

of fatigue life should strictly be linked to material properties (not to specimen geometry) and be method-independent. Further analysis did not completely confirm this last proposition (Pronk [40]).

In practice, the use of graphical method is very subjective. Rowe's method (Rowe and Boulidin [45]) finally allows for a precise mathematical definition of the point, where the dissipated energy ratio as a function of load cycles diverts from a straight line. In stress controlled mode the reduced dissipated energy ratio was introduced according to the formula

$$R_n^\sigma = n \cdot E_n \quad (2.10)$$

where:

n number of loading cycle,
 E_n stiffness module in n -th cycle, MPa.

The reduced dissipated energy ratio concept is illustrated in Fig. 2.9. The number of load cycles on the X-axis correspond to the maximum point of the graph represents the new fatigue criteria. In controlled strain mode the reduced dissipated energy ratio is expressed by

$$R_n^e = n/E_n \quad (2.11)$$

where:

n number of loading cycle,
 E_n stiffness modulus in n -th cycle, MPa.

Using this formula for determining fatigue life N_1 is not as easy as in the previous stress controlled situation. It was however proven that in this mode Eq. (2.10) may be applied and a similar graph may be developed (Fig. 2.10). Hence, fatigue life N_1 may be calculated in a similar way.

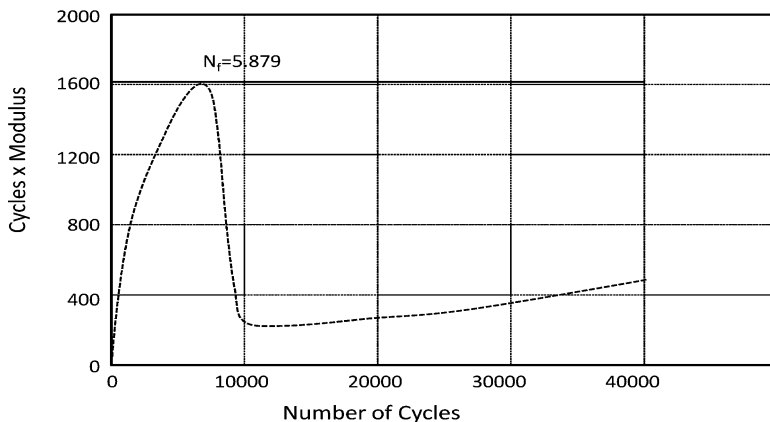


Fig. 2.10 Reduced dissipated energy ratio R_n^E

2.1.2 Review of Asphalt Binder Fatigue Test Methods

2.1.2.1 Historical Development of Asphalt Binder Fatigue Test Methods

Pell Device

The problem of fatigue of road pavements and asphalt mixtures has been a topic of interest and research for several decades. First, pioneer tests and experiments were carried out by Pell [33] at Nottingham University in 1950s. This work was inspired and financed by Shell. Pell in his tests included asphalt mixtures, mastics and binders. Tests for mixtures were carried out with a specially constructed testing machine using a bending method. The apparatus is shown in Fig. 2.11.

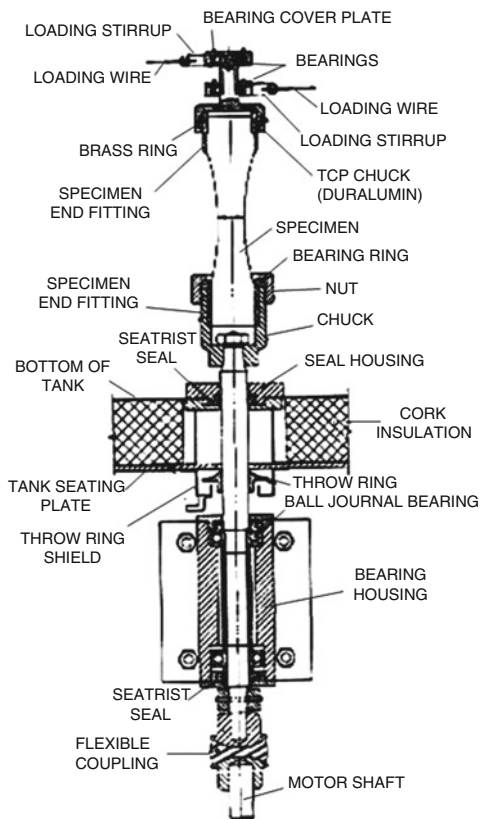
Fatigue tests for mastics and asphalt binders were carried out with equipment, specially constructed for this purpose (Fig. 2.12) and designed for rotational tests.

In subsequent years researchers concentrated on fatigue testing of asphalt mixtures. Studies on binder fatigue have not been carried out for many years. The reason for such a situation was the high complexity of binder fatigue tests.

Fatigue with Fraass Apparatus

In the beginning of 1990s, Potschka carried out studies on binder fatigue using the Fraass apparatus for bending tests (Potschka and Schmidt [36]). Since the bending was given by the Fraass test set-up, tests were conducted at only one loading strain. In this study PmBs and base bitumen were tested at -10 , -15 and -20°C . Some PmBs were also tested after natural weathering and after oxidation. These studies show large differences between different base binders depending on the crude origin. The test results also show that in general PmBs have better fatigue

Fig. 2.11 Equipment for fatigue testing with bending method (Pell [33])



resistance. However, the effect of aging or oxidation is not clear from this paper; one binder improved while a second binder deteriorated after aging. Regarding the test conditions, the authors advise to perform at least three repetitions, even better five repetitions, and in case of large differences between the numbers, the authors advice to use the tests with the best fatigue results as the more correct one.

SHRP – 4 Point Bending

In the frame of the SHRP program, realized in the years 1994–1998, a new equipment for binder testing with a four point beam bending (Figs. 2.13 and 2.14) setup was developed by Anderson et al. [4]. The binder sample was put on a flat aluminum bar creating a composite sample (Fig. 2.15). This sample was subjected to cyclic bending under controlled strain (displacement) in cycles with 0.1 s loading and 0.9 s unloading (rest period). Tests were conducted at four strain levels. For each strain level three repetitions were used. Tests were terminated when hairline cracks became visible at the surface of the binder sample. Studies were made for unmodified standard binders only (at the beginning of SHRP program in

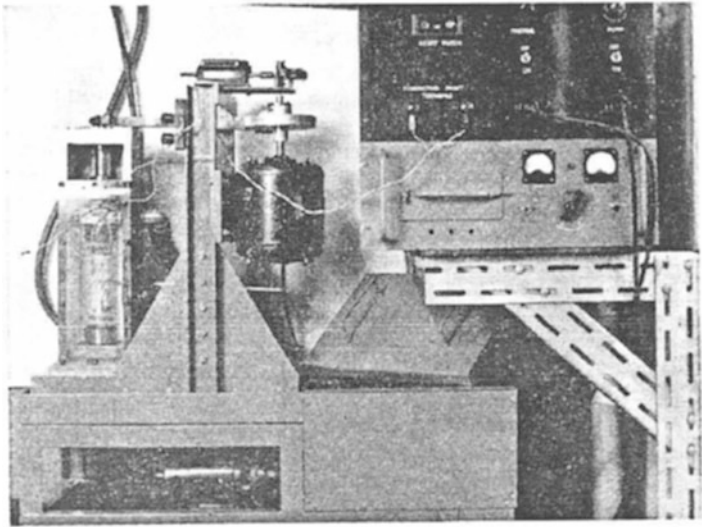


Fig. 2.12 Equipment for fatigue testing using rotational method in Pell’s studies [33]

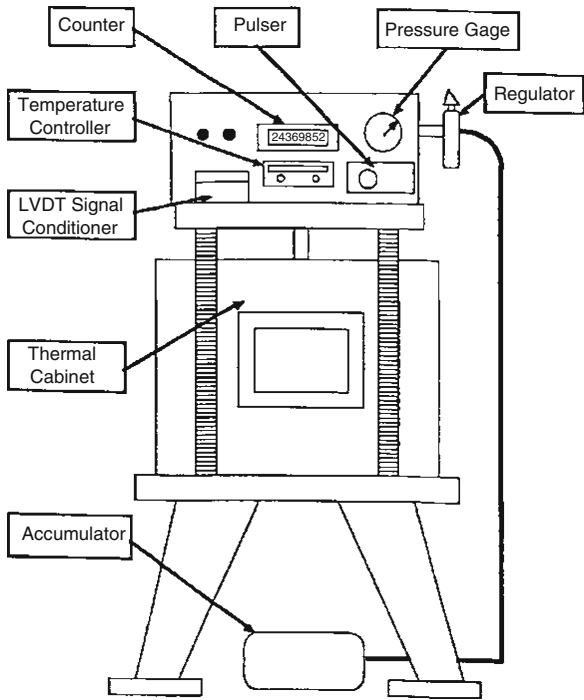


Fig. 2.13 Equipment for fatigue testing using four point beam bending method developed in the SHRP program

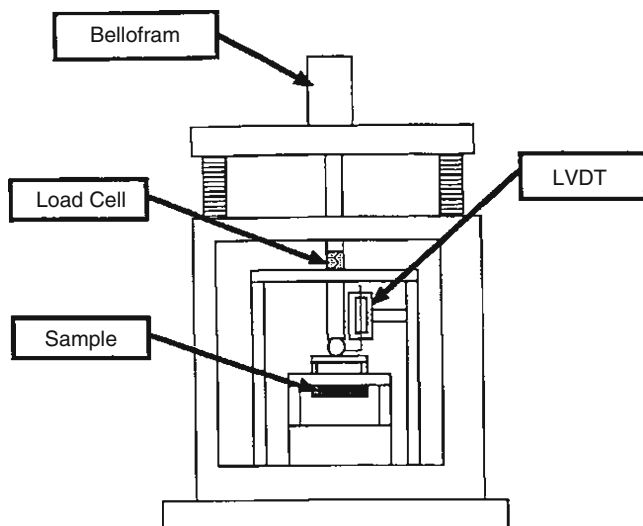


Fig. 2.14 Diagram of specimen loading conditions in fatigue test according to SHRP



Fig. 2.15 Composite specimen of asphalt binder at aluminum flat bar for fatigue testing

USA polymer-modified asphalts were not applied). In these tests, binder fatigue data were measured between 0 and -30°C . Tests at -30 and -20°C showed very little difference between the SHRP core binders; at 0°C the differences were larger. The binders were also investigated after aging. However, aging seemed to have very little influence on the fatigue result.

In the second half of 1990s, in Europe, more attention focused on binder fatigue tests. One example is the research conducted by BP (Claxton et al. [16]). The main goal of this research was to look for correlations between the fatigue behavior of asphalt mixtures and the rheological properties of road binders. It was found that it is possible to get reliable relationships between rutting resistance, low-temperature cracking and rheological properties of asphalt binders, but there is no such relationship for fatigue. The authors indicate that there should be a method to evaluate the fatigue of binders directly, but they did not develop such a method at that time.

DSR

In 1999, several papers were published that used a standard dynamic shear rheometer (DSR) to investigate binder fatigue properties (e.g. Bahia et al. [10];

Phillips [34]; Soenen and Eckmann [46]). In these papers fatigue is measured by continuing load cycles until a decrease in the evolution of the complex shear modulus G^* versus loading time occurs. Bahia and Soenen used the plate-plate set-up, while Phillips used cone-plate geometry. Bahia conducted tests in the linear and also in the nonlinear range of visco-elasticity with fixed number of load cycles (5,000 or 11,000 with strains of 1, 10, 20, 50%). Material failure was defined by the amount of the reduction of G^* . In these studies, estimation and measurement of asphalt healing was also considered. It was shown that both strain dependency and fatigue are highly sensitive to the composition of the binders, the type of additives, the temperature, the heating rate, the aging process and the interaction of these factors. A significant improvement of fatigue properties can be achieved through binder modification with polymers. The papers indicate that high strain testing and determination of nonlinear properties are essential for understanding the mechanical role of binders in asphalt mixtures. The study of Phillips [34] did not show any improvement of the fatigue behavior by styrene-butadiene-styrene SBS modification, but demonstrated that SBS modification could accelerate healing.

In the study of Soenen and Eckmann [46] the influence of binder oxidation and binder modification was presented. Polymer modification significantly improved binder fatigue resistance. Oxidation increased the slope of the fatigue line, thus only creating improvement for low strain levels. In this paper, difficulties in testing and limitations of the method are also discussed. At the Eurobitume workshop 1999 it was observed that binder fatigue tests appear promising for studying the mechanism of fatigue and healing (Bonnot et al. [14]) However, it was also revealed that the mechanisms and properties driving fatigue and healing of binders are still subject to hypothesis, numerous and not acting in a simple way. It was also stated that “a fatigue test on the binder alone can be relevant to characterize the contribution of the binder to the fatigue resistance of the mix, if fatigue involves a cohesive failure but not if it involves adhesive failure.”

It was also stated that, with regards to fatigue cracking, finding performance related binder specifications would imply to answer the question: what properties of the binders are related to the fatigue performance of asphalt layers in a pavement? There is no unique answer to this problem because the required properties of the binder are different depending on the layer and its role in the pavement. The optimal properties are different for a binder in a thin layer with high deflections and in a thick asphalt layer with small deflections.

A lot of factors become important, such as: location of pavement layer, different requirements for pavement construction connected to the expected type of loading. During laboratory tests a lot of difficulties appear, particularly regarding simulation of loading conditions or simulation of healing phenomena.

LCPC Repeated Local Fracture of Bitumen Test

At about the same time a new test device, the Repeated Local Fracture of Bitumen Test, was developed at LCPC (De La Roche et al. [27]) to evaluate repeated fracture and healing phenomena in a thin binder film. In this test, a thin layer of binder

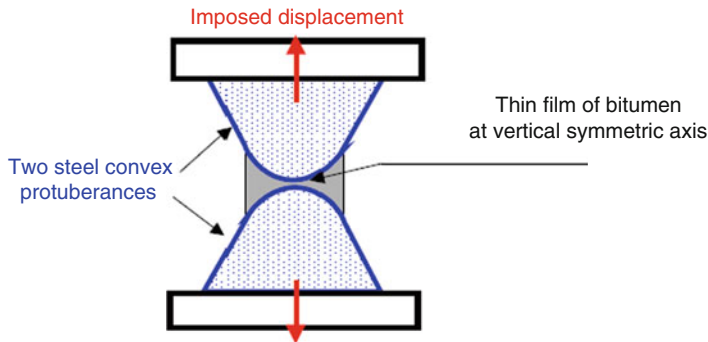


Fig. 2.16 Setup of repeated local fracture of bitumen test (LCPC [27])

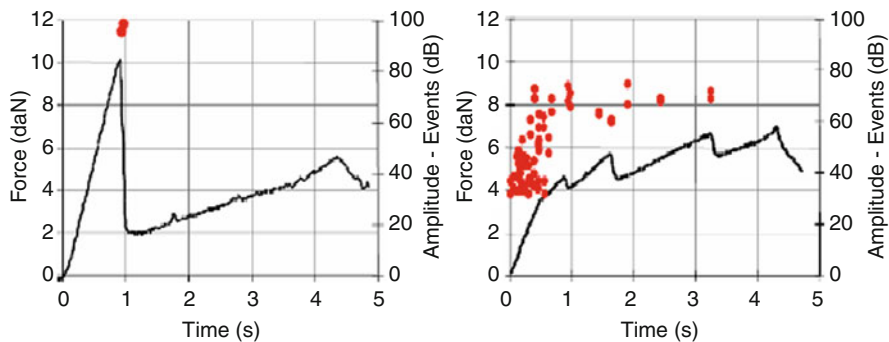


Fig. 2.17 Comparison of repeated local fracture of bitumen test with acoustic emission results (LCPC [27])

placed between two steel hemispheres (simulating aggregate grains) is subjected to cyclic and constant controlled tension strains (Fig. 2.16).

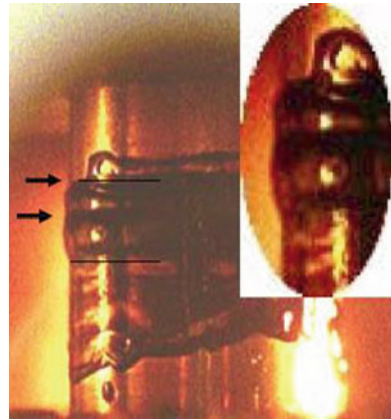
During the test, an acoustic emission technique is used to determine what happens inside the sample. A good correspondence between the force versus time behavior and the acoustic emissions could be observed (Fig. 2.17). Figure 2.17 is also an illustration of two types of crack development: single or multiple (repeated).

The tests showed a wide variety of binder behavior (brittle, ductile) depending on temperature and strain amplitude. Breaking between subsequent loading steps allows observation of the binder healing phenomena.

2.1.2.2 Binder Fatigue Test Using a DSR: Achievements and Limitations

Within the last decade many research centers have started studies on binder fatigue phenomena (Bahia [6]; Bonnetti et al. [13]; Anderson et al. [3]; Antunes et al. [5]; Hammoum et al. [22]; Airey et al. [1, 2]; Planche et al. [35]). The goals are multiple, e.g.:

Fig. 2.18 A picture of the edge effect during fatigue testing in DSR (According to Anderson)



- understand the binder component in the fatigue of the asphalt mixture,
- be able to measure an intrinsic fatigue behavior of the binder itself,
- compare and rank binders with regard to fatigue performance,
- evaluate the influence of binder modifications or additives, without having superimposed effects of the mix design or bitumen filler interactions,
- evaluate the healing capacity,
- develop a surrogate test, and finally also
- work out a reliable method for asphalt binder testing, which could possibly be a part of a new standard.

Binder testing is much cheaper and less labor-consuming than mixture testing. But in order to be useful, one needs to be sure that binder fatigue tests are related to the field behavior of asphalt layers under traffic loading.

Regarding DSR for binder fatigue testing, researchers have also identified the main problems and limitations of these techniques.

One of the main problems is related to so-called edge effects (also called edge failure) which are common for high viscosity samples like polymer melts. In case of such effects, the sample appears to cut in at the mid-plane and to flow out closer to the plate surfaces (Figs. 2.18 and 2.19). As a consequence the registered value of the complex modulus G^* is decreased. Edge effects occur faster with increasing shear rate. They can be reduced if lower gaps between the parallel plates (or also cone-plates) are used. It has been observed by Soenen et al. [48], that the fatigue behavior becomes independent of the gap setting for high stiffness levels which means that edge effects do not interfere in this case. Edge effects can also be avoided by decreasing the test temperature, and in doing so, by increasing the stiffness of the binder sample. At sufficiently high stiffness levels, edge effects are no longer visible (Fig. 2.20).

A second problem is related to the high stiffness levels often used in DSR testing for measuring fatigue properties. As discussed before, these high stiffness levels are needed to avoid edge effects. At these very high stiffness levels compliance errors start to occur. This means that not all the observed deformation is caused by the

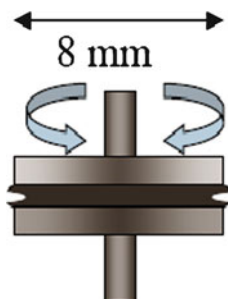


Fig. 2.19 Illustration of edge effect in DSR

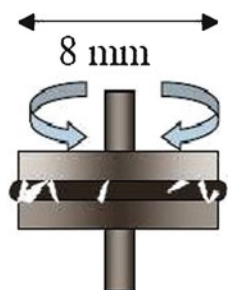


Fig. 2.20 Illustration of proper fatigue failure in DSR test

sample but is partly coming from the metal pieces in the instrument. Since the measured deformation is larger in this case than the true deformation, this results in modulus levels that are too low. One possibility to check for these effects is to measure and compare stiffness levels for different geometries. Only the parts that overlap are correct measurements. Such a test is illustrated in Fig. 2.21. Here, overlapping of the data measured with two DSR plate geometries (8 and 25 mm) is only satisfactory at a phase angle above ca. 72° .

Another possibility to find compliance errors is to measure a known material, for example in Fig. 2.21. When the phase angle in the Black diagram tends zero, the complex shear modulus should approach 1 GPa in this case. Note the deviation of the straight line to the data points at the maximum stiffness level of 0.1 GPa that is still measured correctly. Note also the deviation in stiffness between this maximum stiffness level and the maximum stiffness level that are obtained from measurements with the 25 mm plate (deviation indicated by the shaded area). Typical for compliance errors is also that the measured phase angle will pretend to have a more elastic behavior than the true phase angle. Because of these two effects, edge failure and compliance deviations, the stiffness range for fatigue measurements with DSR is very limited.

Another aspect that needs to be addressed very carefully during binder fatigue testing is the adhesion of the binder to the plates during the whole test. If, at a

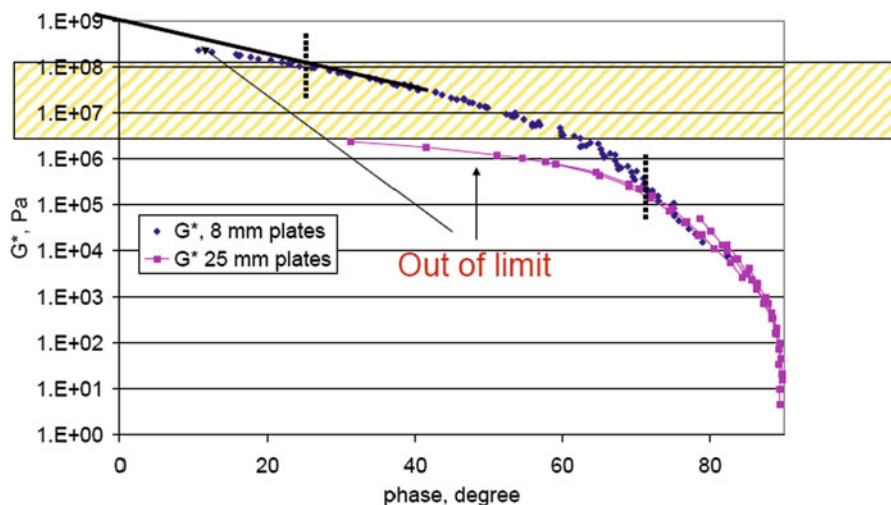


Fig. 2.21 Effect of apparatus compliance at low temperature for binder tests in DSR

certain point, the bitumen is released from the plate, this will, of course, give the impression of a fatigue failure while it is not. Good adhesion can be achieved by increasing the sample temperature to a temperature above or close to the Ring and Ball RB softening point while attaching the sample to the DSR plates.

In the work of Bodin et al. [12] the bitumen temperature was measured carefully during a fatigue experiment. It was observed that the temperature can increase by 0.9°C at the start of the fatigue test, leading to lower modulus values. This suggests that the drop in stiffness observed at the onset of a binder fatigue test may be partly caused by the increase in temperature and partly by non-linear effects inside the binder.

There are still other parameters that can interfere during a fatigue test; one of them is physical hardening. This means that the binder slowly increases in stiffness isothermally. This effect can continue for a long time (days and even weeks) and influence the fatigue test results, sometimes even by producing an increase instead of the expected fatigue induced decrease in modulus. Examples of this phenomenon and other interferences are available in literature (e.g. Planche et al. [35]; Lu et al. [30]).

Remarks on Binder Fatigue Test Using DSR

From the previous sections, it is clear that there is a lot interest to evaluate binder fatigue properties and there are also indications that fatigue phenomena are taking place inside the binder. However, there are still doubts about the relevance of such a test and about its relation to mix fatigue and more significantly to field fatigue behavior.

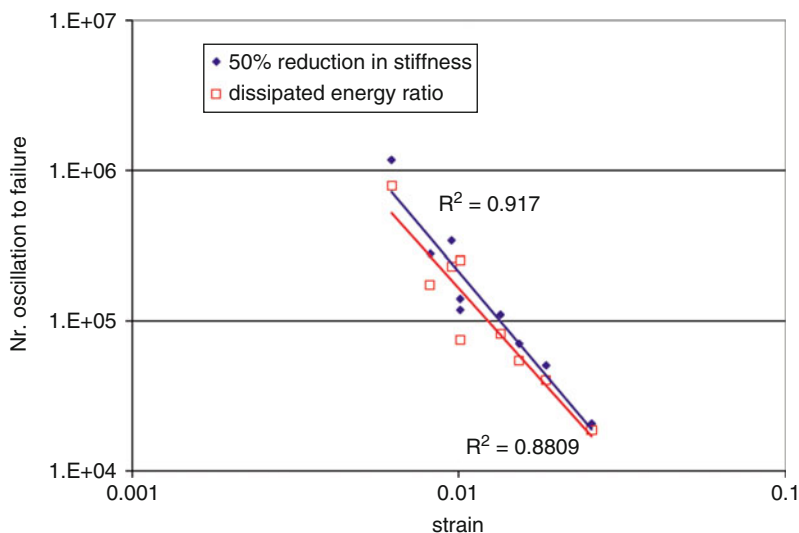


Fig. 2.22 Comparison of binder fatigue characteristics from conventional and dissipated energy methods (Bahia [8])

It also follows from the methods presented above that most commonly DSR shear fatigue tests are performed. Hence DSR fatigue has the potential to become widespread because of common access to DSR rheometers in research laboratories and during the next few years probably also in control laboratories. This is due to the finalized SHRP program where tests for rheological properties (stiffness modulus and phase shift angle) were developed and introduced as the new standard for road binder grading in USA. SHRP was also a strong impulse for using such methods in Europe, where the new standards for road binder are prepared.

Previous experiences with DSR binder fatigue lead to some important observations, which need to be taken into consideration during the preparation of further research projects (Bahia [8, 9]). Among others they are:

- analysis of test results may be carried out conventionally (assuming that specimen failure occurs when G^* has reached 50% of its initial value) or with dissipated energy method (determining changes of dissipated energy in each loading cycle) (Figs. 2.22 and 2.23),
- during binder fatigue tests it is possible to distinguish three phases (Figs. 2.24 and 2.25): first phase with a drop of binder stiffness modulus G^* due to temperature increase; second phase with slow modulus G^* decrease because of material weakening; third phase with a faster modulus G^* decrease as a result of micro-crack development,
- tests may be conducted in controlled stress or strain mode (Figs. 2.26 and 2.27);
- influence of thermal history and binder aging should be considered;
- attention should be paid to avoid edge effects (reduce the temperature to allow stiffer specimens) or add filler to obtain a stiffer sample

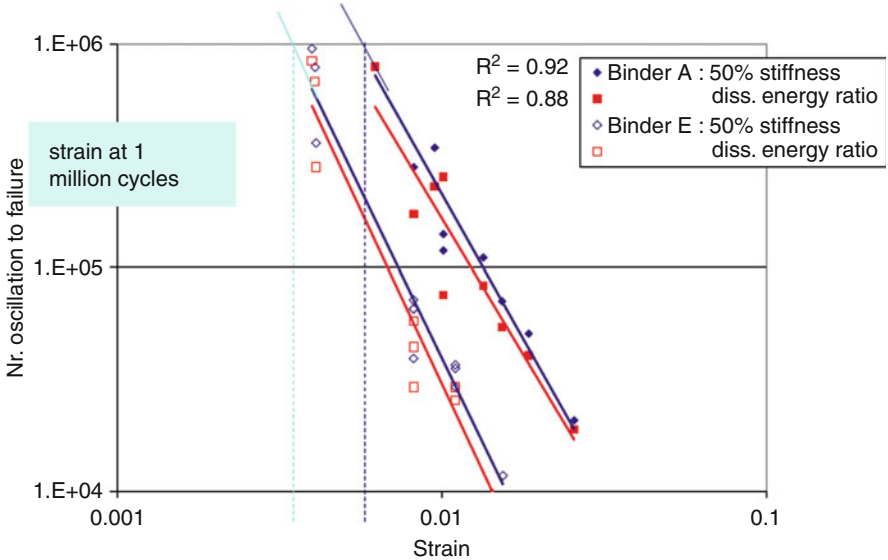


Fig. 2.23 Comparison of fatigue characteristic for two different binders for conventional and dissipated energy methods (Bahia [7])

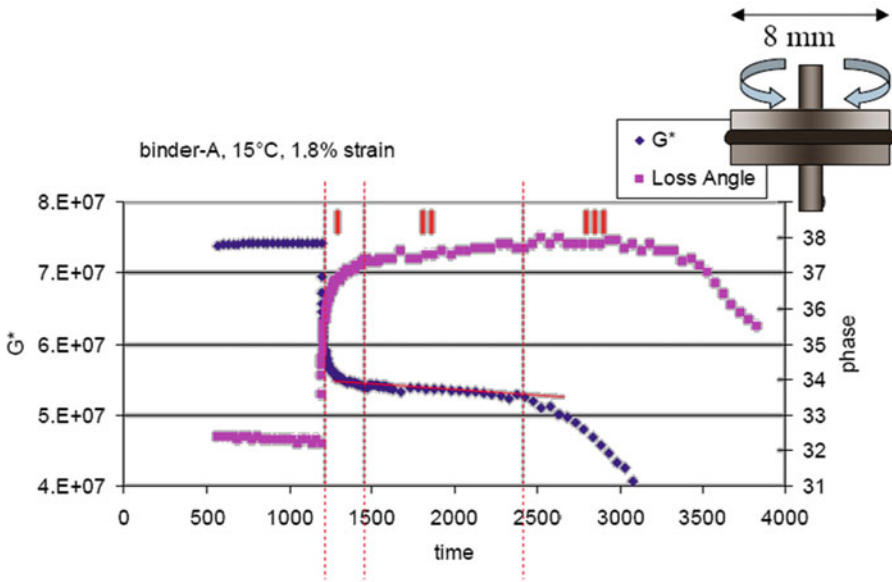


Fig. 2.24 Binder fatigue development due to large strain shear fatigue in DSR (Bahia [7])

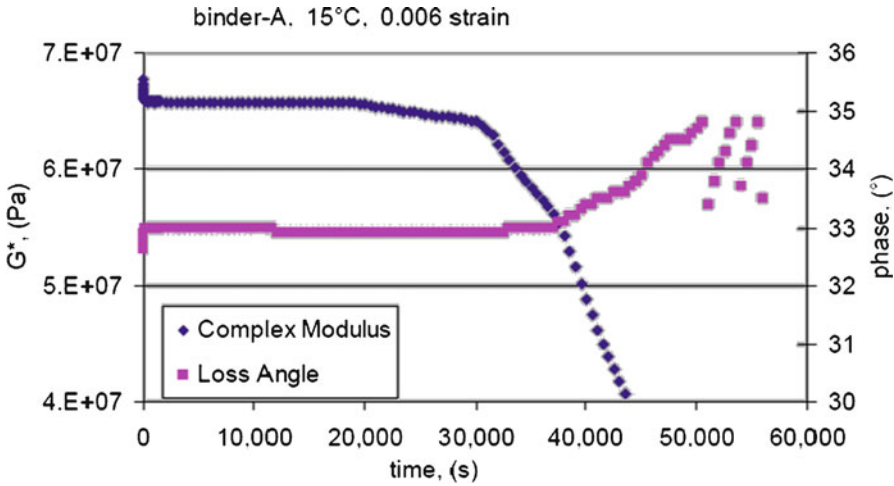


Fig. 2.25 Binder fatigue development due to small strain shear fatigue in DSR (Bahia [7])

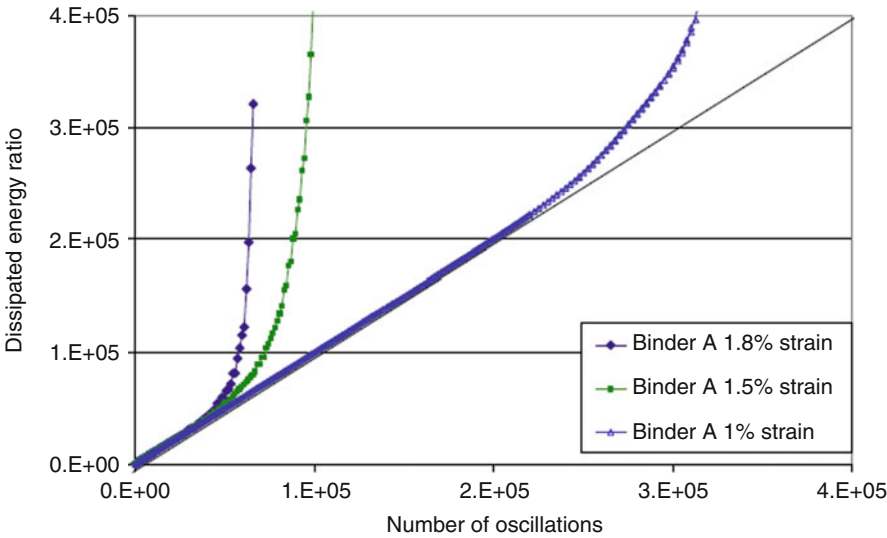


Fig. 2.26 Binder fatigue test in DSR with controlled strain mode (Bahia [7])

- as for standard stiffness testing, attention should be paid to the test temperature and good calibration of this temperature.
- In addition, and specifically for fatigue testing, attention should be paid to changes in temperature during the fatigue experiments.
- Finally the apparatus stiffness (compliance) should be considered (Fig. 2.21).

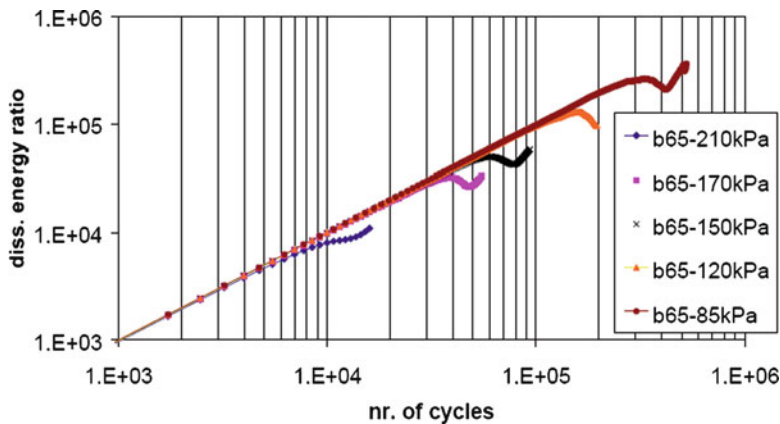


Fig. 2.27 Binder fatigue test in DSR with controlled stress mode, 15°C and 50 Hz (Bahia [7])

Among other important research projects dealing with asphalt binder fatigue testing, one was initiated by the RILEM Technical Committee TC 206-ATB. The project denoted as a RRRBFT (RILEM Round Robin Binder Fatigue Test) was prepared by Task Group TG1-Binders. The details are described Sect. 2.2.

2.1.2.3 Correlation Between Fatigue Test for Binders and Asphalt Mixtures - Rotational Method for Mixes

Regarding the relation between binder and mix fatigue tests comparatively few studies exist. However, they often revealed similarities in the fatigue behavior of binders and to mixtures. For example, as for mixtures, the binder fatigue tests results can also be subdivided into three phases.

In a research project by Bahia [7] the correlation between results from fatigue test obtained in DSR and rotational fatigue tests for sand-asphalt mixtures were investigated. In Figs. 2.28 and 2.29 such relationships are shown. Results before shifting between both methods are shown in Fig. 2.28 and results after shifting using transformation coefficients are presented in Fig. 2.29. These charts show that a clear correlation between these two methods exists allowing to compare results from both methods.

Similarly, if one plots the numbers of cycles to failure versus the strain level, a straight line on log-log scale is obtained. As in mixture tests other failure criteria like dissipated energy can be applied showing behavior similar to the one observed for binder tests. Finally, a temperature increase like in mixture tests also appears in binder fatigue tests.

There are some studies available where binder fatigue tests using the DSR were directly compared to mixture fatigue tests. In a paper by Soenen et al. [48], for example, DSR tests were compared to two point bending fatigue tests of the corresponding mixtures. Test temperature and frequency were the same in both

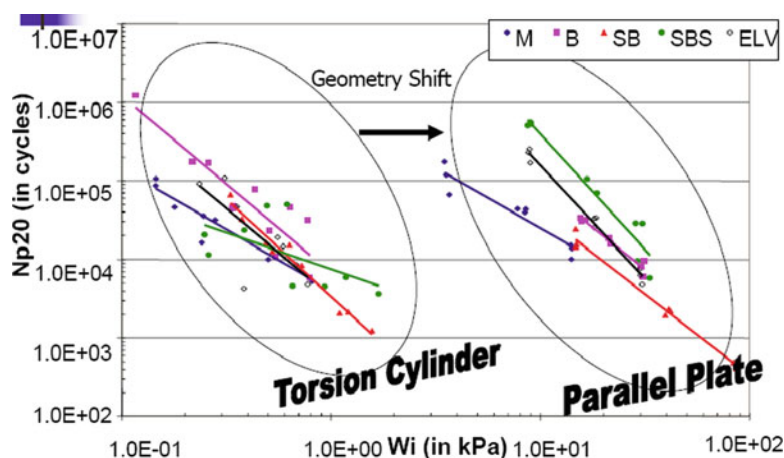


Fig. 2.28 Binder fatigue test results obtained from DSR and rotational test for sand-asphalt mixture before shifting (Bahia [7])

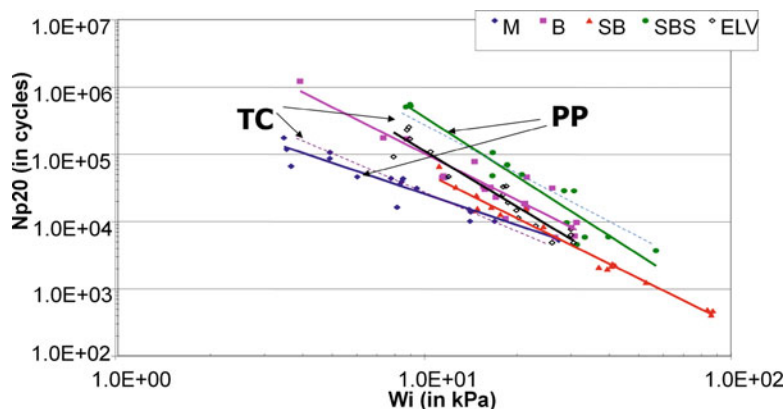


Fig. 2.29 Binder fatigue test results obtained from DSR and rotational test for sand-asphalt mixture after shifting (Bahia [7])

tests. In this study, a good relation was found for unmodified binders, while for two styrene-butadiene-styrene SBS modified binders the fatigue resistance in mixture tests was not as good as predicted from the binder tests.

A lot of work in this field was conducted by Bahia [7], and a short research description is presented in next paragraph together with development of other methods for mixture fatigue tests.



Fig. 2.30 Metravib apparatus for rheological tests of asphalt binder in LCPC

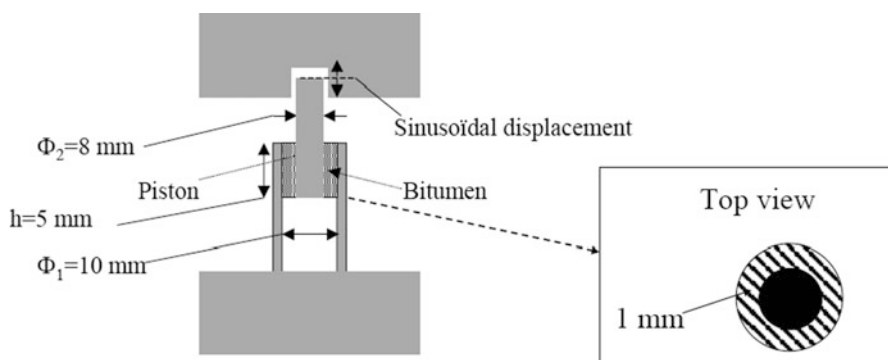


Fig. 2.31 Diagram of binder fatigue test with ring shearing method

2.1.2.4 Development of Other Test Methods for Binder and Mixtures Fatigue

In this section, recently developed tests methods other than the DSR test for measuring binder or mastic fatigue are presented.

Metravib Based Tests: LCPC Experience

In addition to the Repeated Local Fracture of Bitumen Test presented in the previous section, other fatigue tests have been developed by LCPC using the Metravib apparatus (Fig. 2.30). This device allows performing tests in annular shear or tension-compression mode of loading. The annular shear method is presented in Fig. 2.31, and some results in Fig. 2.32. Nevertheless, this test is no

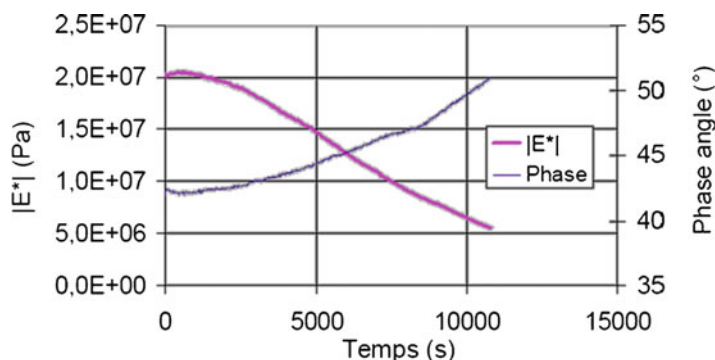


Fig. 2.32 Test results of binder fatigue test with ring shearing method

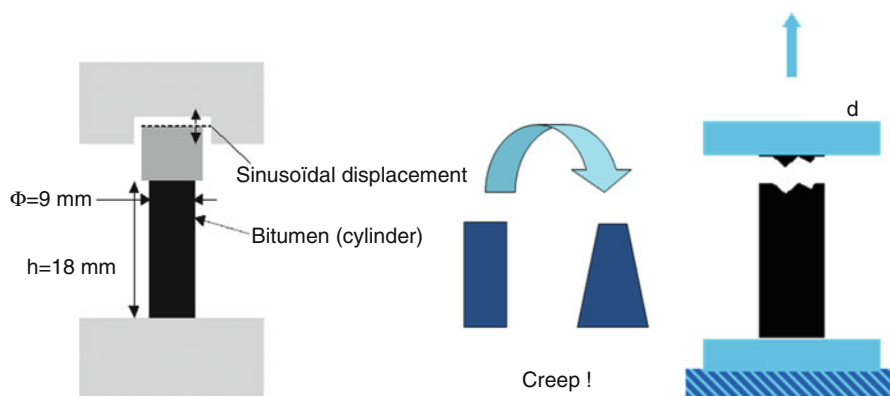


Fig. 2.33 Fatigue test in tension-compression load mode performed on cylinder specimens disturbed by creep and adhesive fracture (Chailleux et al. [15])

longer used due to the mode of damage mechanism which occurs at the interface between steel and binder and not in the binder itself.

A tension-compression method has also been studied. The first experiments were conducted on a cylinder (Fig. 2.33), but problems occurred due to adhesive fractures between bitumen and steel at low temperature (ca. 10°C) and creep flow phenomena (Fig. 2.34) at higher temperature (ca. 20°C).

This problem was solved by choosing a geometry where the cracking zone was controlled (Chailleux et al. [15]). A diabolo shaped sample was designed to give a uniaxial stress state under tension and to allow concentrating stresses in the material itself (Fig. 2.34). The samples are manufactured by pouring hot bitumen (160°C) into a silicon mould. When room temperature is reached and in order to obtain a perfectly flat surface, the upper part is trimmed with a hot knife. The sample is then stored in the mould at low temperature (−10°C) until testing. As soon as the sample

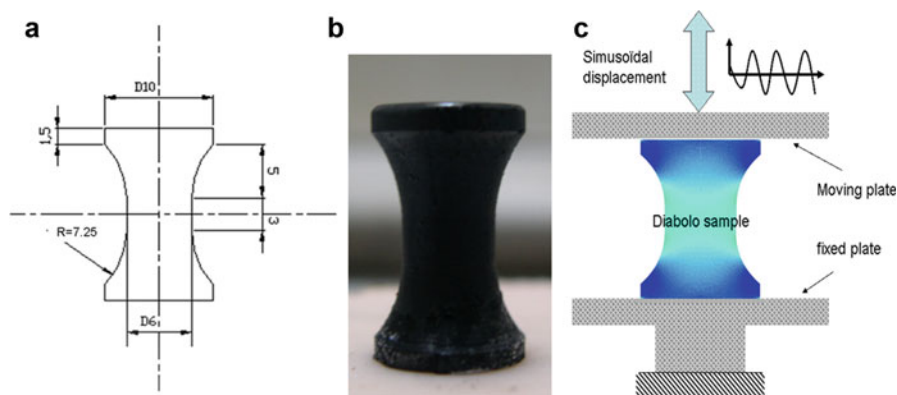


Fig. 2.34 (a) Dimensions of the Diabolo specimen (mm), (b) Diabolo specimen after demoulding, (c) Schematic view of the sample loaded in alternative tension compression mode (Chailleux et al. [15])

is demoulded (Fig. 2.34b), it is glued onto the two parallel plates with a Cyanoacrylate adhesive (Fig. 2.34c).

With this new geometry, stiffness during sinusoidal loading of bituminous material shows classical fatigue behaviour (Fig. 2.35a). Just after the conditioning time, a rapid decrease of the stiffness is observed (Phase 1), followed by a slow damaging process (Phase 2). At the end of the fatigue process, a dramatic decrease of the stiffness due to specimen failure occurs (Phase 3). Figure 2.35b shows cracking in the material, at the shoulder which is in accordance to the finite element analysis.

Torsion Cylinder

The edge effect observed during DSR shear testing promoted the search for different fatigue binder test methods in the USA. One suggestion is to make a rotational test for asphalt-sand mixture. Tests of this kind are performed at the University of Wisconsin in Madison by Bahia's team.

A sketch of the test together with characteristic failure mode is shown in Fig. 2.36. It is worth noting that this test refers to Pell's pioneer work [33].

In this test, a mixture of bitumen (binder) and sand is used. The sand comes from one source, i.e. from Ottawa's neighborhood in Canada. In Fig. 2.37 the mould and the way of specimen preparation is presented.

In Figs. 2.38 and 2.39 the rotational test and a specimen failure are shown. It is very important to obtain a failure crack near half the height of the specimen and not at the ends, where stress concentrations are located. In Fig. 2.40 an example test result is presented illustrating the decrease of material stiffness modulus and increase of phase angle.

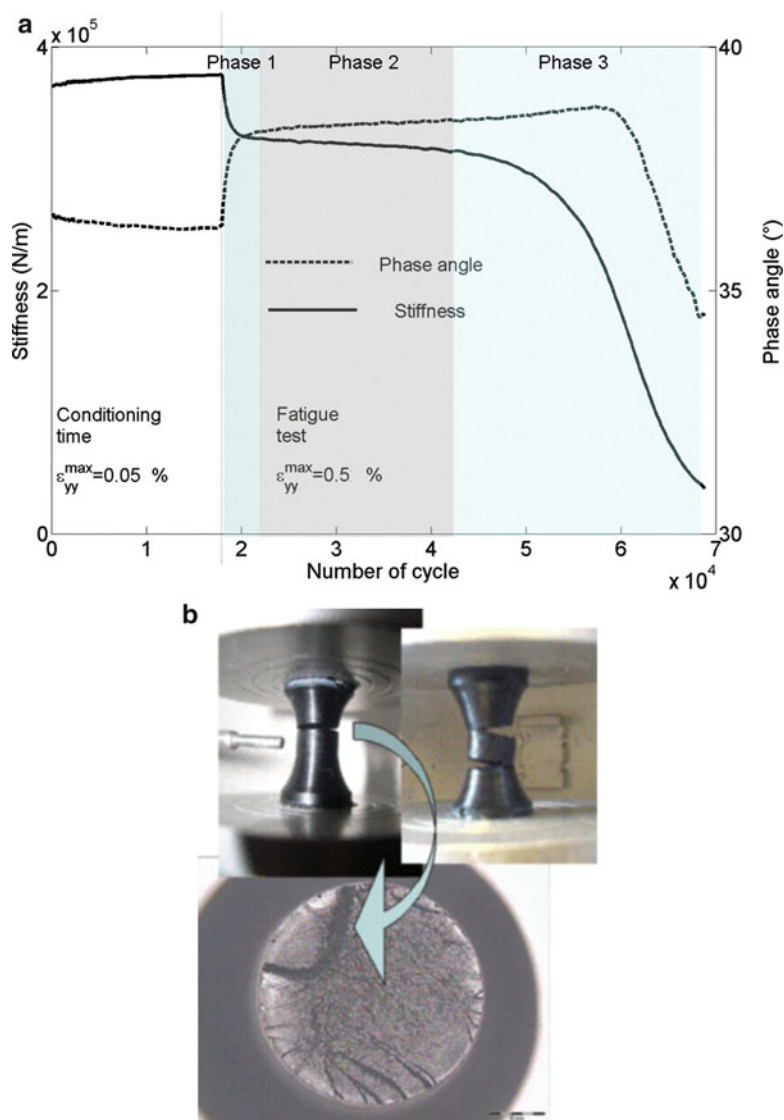


Fig. 2.35 (a) Stiffness and phase angle, plotted versus the number of cycles during the conditioning time at low strain and during the fatigue test at 0.5% strain. (b) picture of the diabolo sample at the end of the fatigue test (Chailleux et al. [15])

Annular Shear Rheometer: ENTPE Experience

Di Benedetto's team at ENTPE (Delaporte et al. [18]) developed the ASR (Annular Shearing Rheometer) for assessing binder and mastic fatigue as well as rheological properties using a classic hydraulic press. A sinusoidal shear stress or sinusoidal

Fig. 2.36 Fatigue test for specimen made of sand-binder mixture with rotational method

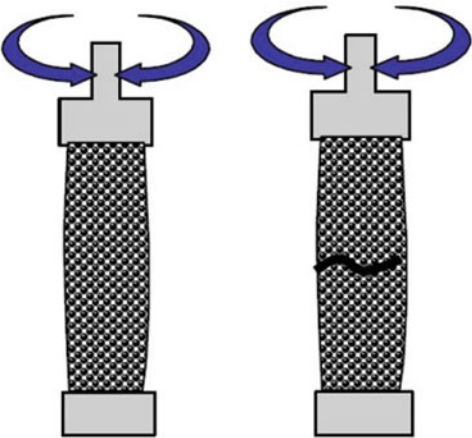


Fig. 2.37 Preparing specimen of sand-asphalt mixture for fatigue testing with rotational method (Bahia [7])





Fig. 2.38 Fatigue testing with rotational method (Bahia [7])



Fig. 2.39 Specimen failure after fatigue test with rotational method (Bahia [7])

shear strain (distortion) is applied on a hollow cylinder of bitumen or mastic at different temperatures and frequencies. The hollow cylinder has a rather large size: 5 mm thickness, 105 mm outer diameter and 40 mm height. With these geometrical features, the test is homogenous as a first approximation even with aggregate sizes

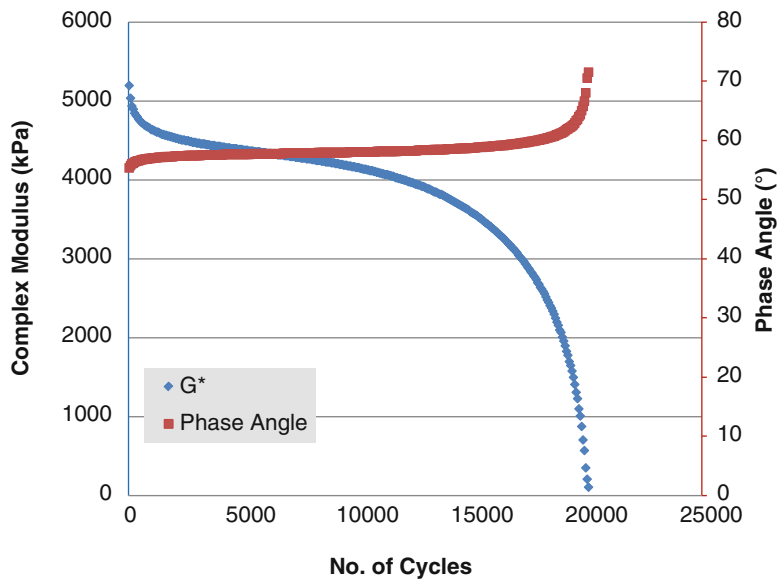


Fig. 2.40 Fatigue test result obtained from rotational method showing that stiffness modulus decreases and phase angle increases (Bahia [7])

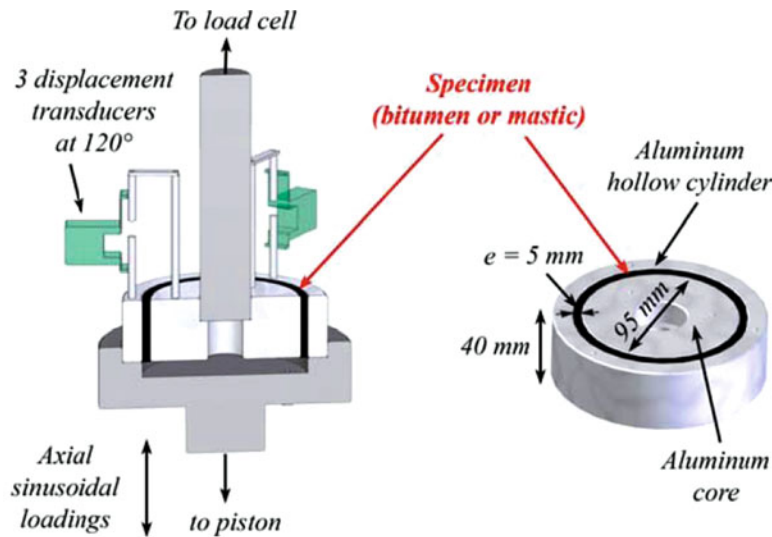


Fig. 2.41 Schematic view of the annular shear rheometer (ASR) for bitumen and mastic (Delaporte et al. [18])

up to 1 mm. A schematic view of the apparatus is presented in Fig. 2.41. An interesting protocol has been defined in which linear viscoelastic measurements are carried out between fatigue sequences (Fig. 2.42). This protocol allows a better

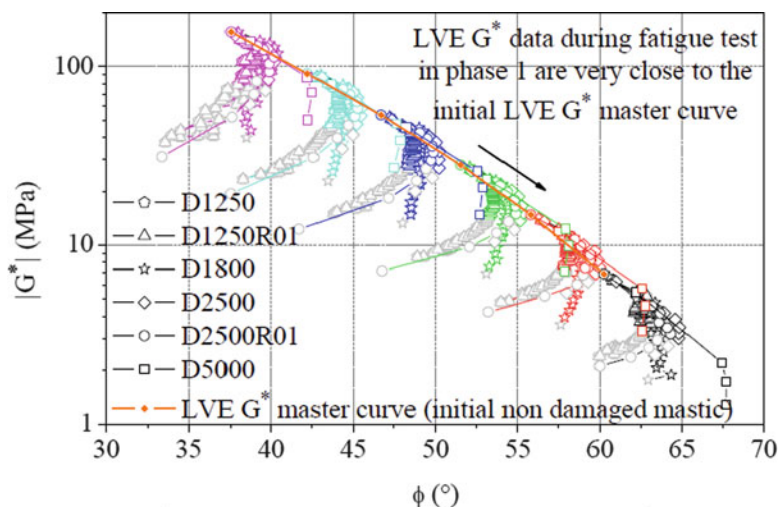


Fig. 2.42 Complex modulus (in the linear domain) measured during fatigue test and master curve obtained with a complex modulus test (Delaporte et al. [18])

understanding of fatigue phenomena. In particular reversible and irreversible damage can be clearly identified. It is also interesting to note that the ASR system allows performing wave propagation analysis at high frequencies (Van Rompu et al. [51]). A continuous assessment of the viscoelastic linear properties can be performed during fatigue experiments.

A similar device, the so called coaxial shear test for binder, has been developed earlier by Junker at EMPA in 1987 ($H = 200$ mm, $D_i = 50$ mm, gap width $e = 10$ mm) and is still used for highly modified polymer bituminous joint sealants (Partl and Hean [31]).

2.1.2.5 Conclusions

Fatigue of asphalt layers is one the most important distress modes in road pavement. Hence, the prediction of asphalt pavement fatigue life is one of the most crucial tasks in the pavement design and has to rely on reliable input obtained from the laboratory tests.

Asphalt fatigue tests of asphalt mixtures have been developed and are already widely applied for quite a long time. This kind of test is relatively time consuming and expensive.

Bituminous binder is one of the most important components of asphalt mixtures and pavements. Hence, development of bituminous binder testing methods and tools are key research areas. In this context, fatigue testing of bituminous binders is an important task with considerable research potential.

This state-of-the art overview presents the main aspects of bituminous binder fatigue, as well as the evolution of research and experimental tools together with the most up-to-date experimental achievements.

2.2 Binder Fatigue Properties and the Results of the Rilem Round Robin Test

2.2.1 Introduction

In the last decades, binder fatigue tests have become common and are used to investigate the fatigue behavior of bituminous binders and mastics, to compare different binder types, and to investigate the role of the binder in mixture fatigue behavior (Bahia et al. [10]; Gauthier et al. [21]; Phillips [34]; Soenen et al. [47, 48]; Planche et al. [35]). The most common equipment for binder fatigue tests has definitely been the DSR plate-plate rheometer, although other test geometries have been proposed by Airey et al. [1]. However, even if this type of testing has become popular there are up to now no data available on the reproducibility of this test.

Task Group 1 (Binders) of the RILEM TC 206-ATB (Advanced Testing of Bituminous materials) undertook the task of organizing a RILEM Round Robin Binder Fatigue Test (RRRBFT). The purpose of the test was to investigate the repeatability as well as the reproducibility of binder fatigue tests.

The Round Robin Test was organized in two phases. The results of the first phase showed some technical problems, which suggested the need of the second phase. The results of the first and second phase of RRRBFT are presented below.

2.2.2 Participating Laboratories

The list of participating laboratories in the first and second phase of the Round Robin Tests is presented in Tables 2.1 and 2.2 (only the participating laboratories which delivered the results of tests are listed). Twenty three laboratories declared participation in the first phase. From those 12 labs delivered results. In the second phase 13 labs declared participation, and 8 delivered results.

2.2.3 Experimental Procedure

2.2.3.1 Samples

Binders

In the first testing phase of the RRRBFT, the investigations were limited to three bituminous binders – two unmodified and one polymer-modified. For the second

Table 2.1 First Phase RRRBFT – participating laboratories and equipment

Name of the participating institution (laboratory)	Name of the responsible person	Type of DSR	Producer of DSR
Shell Global Solutions	Sophie Nigen-Chaidron	ARES/AR100/MCR501	TA Instruments/Paar-Physica
Università Politecnica delle Marche	Francesco Canestrari, Gianluca Cerni	RT10	Haake
Total Research Center Solaize	Gilles Gauthier	RDAII	Rheometrics
Laboratorio De Asfaltos Del Centro Tecnológico De Repsol YPF	Patricia Bernaldo De Quirós Mirabent	Bohlin DSR 50	Bohlin
Ooms Avenhorn Holding, Research & Development	Ronald C. van Rooijen	UDS 200	Paar Physica
Nynas Belgium AB	Hilde Soenen	Paar Physica MCR 500	Anton Paar
NTEC, University of Nottingham	Gordon Airey	CVO or Gemini Dynamic Shear Rheometer	Bohlin
LCPC (now IFSTTAR)	Chailleux Emmanuel	Diabolo tension - compression test	Metravib
University of Wisconsin–Madison	Hussain Bahia	AR 2000	TA Instruments
EMPA	Remy Gubler	Bohlin Rheometer DSR CS	Bohlin Instruments
IBDM	Dariusz Sybilski	Mars II	Haake
Federal Institute for Materials Research and Testing (BAM, Berlin)	Christoph Recknagel	Physica Universal Dynamic Spectrometer UDS 200	Anton Paar Delamination of samples from upper plate Practically no results

phase test, three straight run bitumens of various grades 10/20, 20/30 and 35/50 were chosen (Table 2.3).

The binders were supplied from different producers and differed in behavior in several physical tests. For example, the Black curves showing the different character of the binders tested in both phases are given in Fig. 2.43. Characteristics of PmB *C can be easily found to differ from the straight run bitumens.

It was also noted that binders A and C showed physical hardening at 10°C and 10 Hz (an isothermal increase in stiffness), while sample B did not show any hardening under these conditions.

In order to avoid testing at an equi-stiffness level or at a constant temperature, the binders were selected to have the same stiffness level at 10°C and at 10 Hz (see Table 2.4).

Table 2.2 Second Phase RRRBFT – participating laboratories and equipment

Name of the participating institution (laboratory)	Name of the responsible person	Type of DSR	Producer of DSR
Laboratorio De Asfaltos Del Centro Tecnológico De Repsol YPF	Mercedes Ayala Canales	Haake Rheostress 600	Haake
IBDM	Dariusz Sybilski	Mars II	Haake
Shell Global Solutions	Claire Laurens	ARES/AR100/MCR501	TA Instruments/ Paar-Physica
Nynas Belgium AB	Hilde Soenen	Paar Physica MCR 500	Anton Paar
Ooms Nederland Holding, R&D Laboratory	Radjan Khedoe	Physica UDS-200	Anton Paar
EMPA	Martin Hugener	Physica MCR 301	Anton Paar
Federal Institute for Materials Research and Testing (BAM, Berlin)	Christoph Recknagel	Physica Universal Dynamic Spectrometer UDS 200	Anton Paar
BP Oil Marketing GmbH Bitumen–Global Product Development and Technical Support	Ronald van Rooijen	Bohlin	Bohlin Instruments

Table 2.3 Binders for RRRBFT

Binder's symbol	Binder's grade	Binder's source
Phase 1		
A	50/70	NYNAS
B	50/70	LCPC
C	PmB	TOTAL
Phase 2		
D	35/50	IBDM
E	20/30	IBDM
F	10/20	SHELL

Mixtures

Three mixtures for each phase test: MA, MB, MC, and MD, ME, MF were manufactured at LCPC, France, respectively with the binders A, B, C (Phase 1), and D, E, F (Phase 2). The bituminous mix is a 0/6 BBC (French standard NF 98–133), with a binder content of 6.88%. The grading curve of the mix is given in Table 2.5. This is a continuous graded mix, made of aggregate from “La Noubleau” quarry (microdiorite). The filler came from “Airvault” quarry (limestone). This mix composition was chosen in order to consider the sample homogeneous in comparison to the sample size used for mechanical test. As a consequence, repeatability of the fatigue test with this composition is known to be good. This allows studying the binder effect.

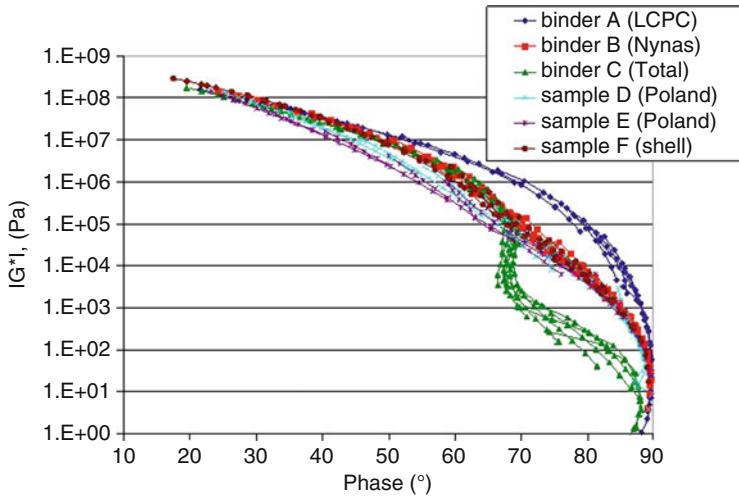


Fig. 2.43 Black curves of the six binders selected for the first and second phase testing

Table 2.4 Overview of some physico-chemical properties of the binders (Phase 1)

	Binder A	Binder B	Binder C
Asphaltene content (%) (defined as insoluble part in n-heptane)	12.4	17.8	17.8
Crystalline fraction (%) (by DSC)	2.23	0.1	2.64
Tg (°C) (by DSC)	−23	−18.5	−25.1
G* (MPa) (10°C, 10 Hz) (average data of RRRBFT, will be discussed further)	48.3	47.7	53.6

Table 2.5 Grading curve of the asphalt mixtures

Sieve (mm)	0.06	0.125	0.25	0.50	1.00	2.00	3.15	4.00	5.00	6.00
% Passing	10.9	14.7	20.1	28.2	39.9	59.9	66.4	69.6	81.1	95.6

Aggregates and binders were mixed in the laboratory according to standard NF EN 12697–35 and compacted with an LCPC plate compactor NF EN 12697–33. Complex modulus measurements were performed by two laboratories: LCPC, Nantes, France and IBDM, Warsaw, Poland. Two mix fatigue test methods were applied, respectively a two point bending (2 PB) test on trapezoidal samples (NF EN 12697–26) in LCPC, and four point bending (4 PB) on prismatic beams (PN-EN 12697–26) in IBDM.

It can be noted in Fig. 2.44, that the viscoelastic behavior of mixes of Phase 1 are quasi-identical, contrary to rheological behavior of binders which are relatively different (especially at high temperature). At 10°C, 10 Hz, mechanical properties of the mix are close to each other as can be seen in Table 2.6. It can be also observed that results from 2 to 4 PB are very close to each other especially with respect to the

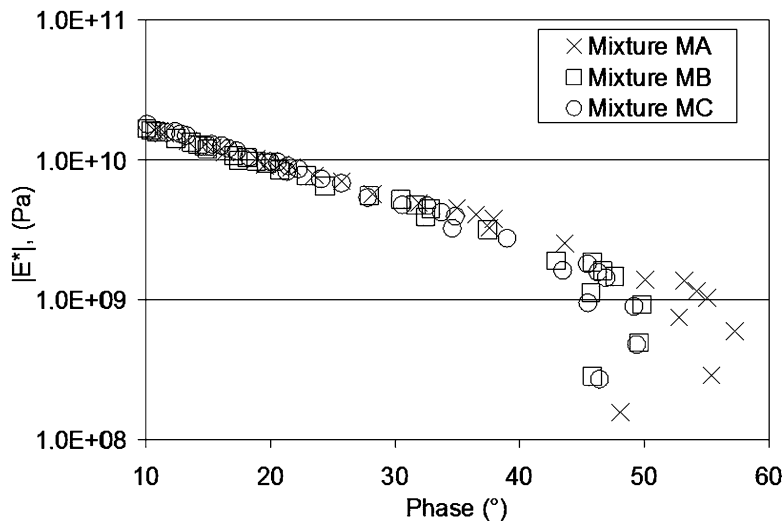


Fig. 2.44 Black curves of the three mixtures prepared with binders A, B and C (Phase 1)

Table 2.6 Mechanical properties of the mixture at 10°C, 10 Hz

	Binder	E* (MPa) (10°C, 10 Hz)	Phase angle (°), (10°C, 10 Hz)	% void
Phase 1				
Mixture MA	A	13,741 (2 PB), 12,193 (4 PB)	13.4 (2 PB), 13.3 (4 PB)	3.6
Mixture MB	B	12,941 (2 PB), 11,606 (4 PB)	15.3 (2 PB), 15.4 (4 PB)	3.2
Mixture MC	C	14,162 (2 PB), 12,282 (4 PB)	12.4 (2 PB), 13.0 (4 PB)	3.4
Phase 2				
Mixture MD	D	10,709 (4 PB)	12.9 (4 PB)	3.3
Mixture ME	E	12,360 (4 PB)	10.8 (4 PB)	3.8
Mixture MF	F	15,805 (4 PB)	8.5 (4 PB)	3.5

phase angle. Stiffness from 2 PB is 12–15% higher than for 4 PB, but the ranking of the mix stiffness is the same.

2.2.3.2 Binder Fatigue Tests

Phase 1

Almost all participants used the 8 mm plate-plate DSR equipment. Only one laboratory used a diabolo-shaped sample in a tension-compression type of equipment. The DSR devices that were used were all controlled stress rheometers, for example from Anton Paar the MCR 501, the MCR 300, the MCR 301 and the UDS200, from Haake the Rotovisco RT10 and the Mars II, and from Bohlin the DSR II 50 and the Gemini 200.

The test procedure included a sample preparation description and a silicon mold suited for the 8 mm plate-plate setup. The homogenization of the sample (heating times and temperature) and the loading of the sample in the rheometer (temperature at loading, trimming, cooling rate to the fatigue test temperature) were described. The equilibration period before starting the fatigue test was set at 30 min, and a recording of the low strain stiffness and phase angle (also at 10°C and 10 Hz) during this equilibration period was requested. The fatigue testing had to be conducted at 10°C and at 10 Hz, in controlled strain mode, tests at three strain levels, 2.5, 1.8 and 0.8%, were requested, and at each strain level three repetitions had to be conducted. Samples were sent to 26 laboratories and data were received from 11 laboratories.

The results of one of the 12 participating laboratories were excluded due to delamination of the binder samples from the steel DSR plates. This laboratory participated also in the second phase successfully. It was one of the optimistic lessons learned from the first phase.

Phase 2

Eight laboratories participated in Phase 2 (all of them participated in the first phase). Due to the experiences from the first phase, some changes in the testing procedure were included in the second phase. Sample preparation, equilibrium period, test temperature 10°C and frequency 10 Hz remained the same. Strain levels requested were: 2.5, 1.8, and 1.3%. The lowest strain was higher than in the first phase (1.3 instead of 0.8%). The change was proposed due to problems observed in the first phase in several laboratories in testing at the low strain.

Special recommendation for the participants in the second phase was to pay attention to the sample temperature control in the DSR when performing the test. In the first phase it was observed in some testing equipment that temperature control was not sufficient. It was found that Haake DSR provided temperature control in the lower steel plate only (the Peltier system), and there was no temperature control in the upper plate. This was the reason why the actual bitumen sample temperature was higher than requested. The result was a temperature gradient in “z” axis direction in the sample. The G^* modulus at 10°C was lower (Fig. 2.45) or at higher testing temperature (e.g. 40°C) higher.

One of the participating laboratories (D) solved the problem for the second phase, providing the self-made temperature control chamber, which was of course not the best solution for the future.

2.2.3.3 Mixture Fatigue Tests

The fatigue properties of the mixes MA, MB and MC were measured by two laboratories, LCPC and IBDM, according to the standard EN 12697. Two modes of loading were used: four point bending on rectangular beams, and two point bending on trapezoidal samples. Tests were carried out in strain controlled mode.

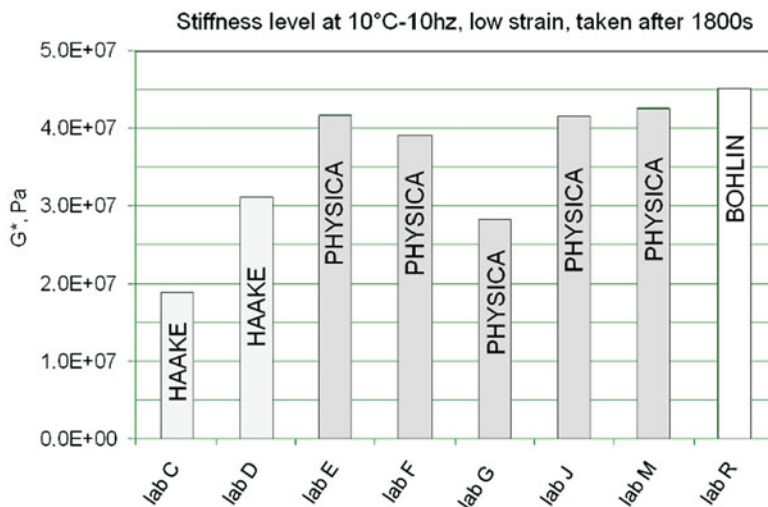


Fig. 2.45 Comparison of G^* depending on DSR type (Phase 2)

Considering the usual scatter of these fatigue tests, a minimum of six samples, for each strain level was used. Fatigue life is defined as the number of cycles which corresponds to 50% decrease of the initial stiffness modulus. The fatigue tests are performed at three strain levels. The lowest strain is chosen in such way that the rupture time reaches one million cycles. Test conditions are the same as the binder fatigue test: 10°C and 10 Hz.

2.2.4 First Phase Results

2.2.4.1 Binders: Low Strain Stiffness and Phase Angle

In this section, low strain stiffness and phase angle, recorded during the equilibration is analyzed. A large number of repetitions are available per sample type and per laboratory, since testing at three strain levels and three repetitions were required. Although in the literature a number of DSR round robin reports already exist, an overview of the results of this dataset is presented.

In Figs. 2.46 and 2.47, the evolution of $|G^*|$ and phase angle in the equilibration period is shown for sample A (only one repetition). The strain is indicated, and based on literature data, all these strains levels are inside the linear viscoelastic region (Airey et al. [1]). For lab-M, the only laboratory that did not use a plate-plate rheometer but a tension compression device, the $|E^*|$ data are divided by three to obtain $|G^*|$ values. Although statistical procedures do not find outliers, some problems become clear: for lab B, thermal equilibrium is not obtained within the 30 min period. The data from lab T indicate a temperature problem, since $|G^*|$

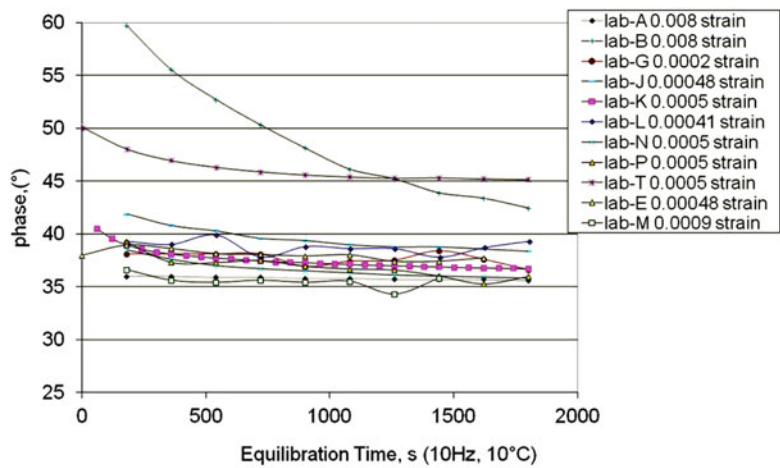


Fig. 2.46 Evolution of the complex modulus norm in the equilibration period (Binder A)

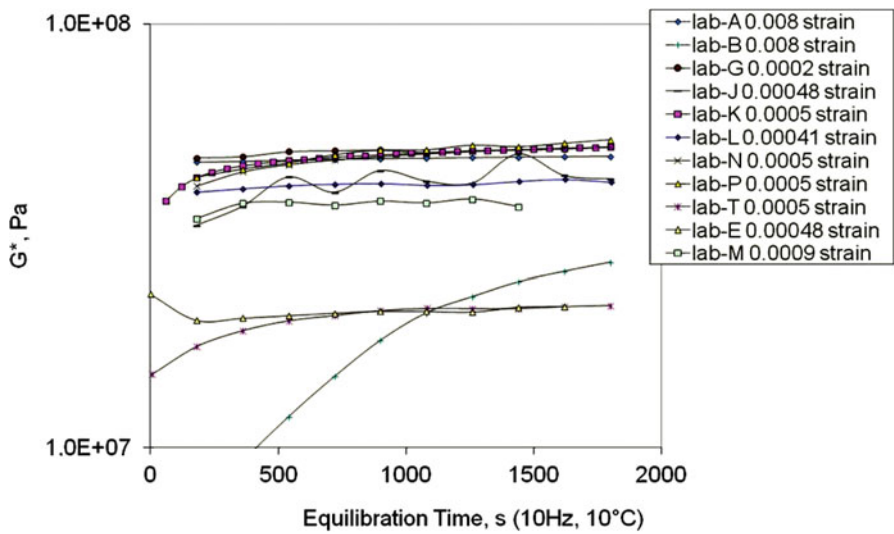


Fig. 2.47 Evolution of the phase angle in the equilibration period (Binder A)

values are slightly lower and phase angles are slightly higher. For lab E, the stiffness seems to be rather low, while the phase angle is similar as for the other laboratories.

Many of the problems indicated in this RRRBFT phase were reported to the labs, and in some of them the problems have been solved. This was important while the second phase test was planned, with the aim to obtain better repeatability and reproducibility levels.

Table 2.7 Overview of average values of low strain $|G^*|$ and low strain phase angle (10 Hz, 10°C, Binder A)

Lab	Average $ G^* $ (Pa)	$ G^* $ COV%	Average δ (°)	δ COV %	Nr, of repetitions	Strain (%)	Notes
A	4.85E + 07	2.1	35.5	1.7	4	0.8	
B	2.39E + 07	8.8	43.6	1.1	9	0.8	No thermal equilibrium
G	4.98E + 07	3.8	37.3	1.9	9	0.02	
J	4.59E + 07	8.2	38.4	1.8	9	0.048	
K	5.18E + 07	3.5	36.4	1.0	9	0.05	
L	4.25E + 07	7.1	38.3	2.2	9	0.041	
N	5.54E + 07	6.7	34.8	2.6	9	0.05	
P	5.32E + 07	4.3	37.0	5.2	9	0.05	
T	2.19E + 07	7.1	44.7	2.5	9	0.05	
E	3.16E + 07	18.6	35.8	4.6	9	0.048	
M	3.94E + 07	4.6	35.3	1.3	8	0.09	$ G^* $ derived from $ E^* /3$

Table 2.8 Overview of average values of low strain $|G^*|$ and low strain phase angle data, for all binders (10°C, 10 Hz), Phase 1

Binder	All data		Excluding data from lab B and T		Excluding data from lab B, T and E	
	$ G^* $ (Pa)	δ (°)	$ G^* $ (Pa)	δ (°)	$ G^* $ (Pa)	δ (°)
Binder A						
Average	4.22E + 07	37.9	4.64E + 07	36.6	4.83E + 07	36.6
StDev	1.17E + 07	3.3	7.55E + 06	1.3	5.44E + 06	1.3
COV %	27.7	8.7	16.3	3.5	11.3	3.7
Binder B						
Average	4.28E + 07	38.9	4.65E + 07	38.0	4.77E + 07	38.4
StDev	1.07E + 07	2.1	7.60E + 06	1.2	7.20E + 06	0.8
COV %	24.9	5.4	16.4	3.2	15.1	2.0
Binder C						
Average	4.80E + 07	34.1	5.20E + 07	32.9	5.36E + 07	33.2
StDev	1.22E + 07	2.9	9.19E + 06	1.3	8.35E + 06	1.2
COV %	25.3	8.5	17.7	4.1	15.6	3.5

In Table 2.7, statistics of the repetitions within each of the respective laboratories are provided for binder A. Presented numbers refer to the average of all repetitions measured within the respective laboratory. The number of repetitions that was provided is indicated. Data relate to binder A only. Coefficient of Variation (COV) was calculated as the average value divided by the standard deviation, (multiplied by 100, if in percentage). The variation between the repetitions, within a fixed laboratory is rather good, for stiffness as well as for the phase angle. The behavior for binder B and binder C is not shown separately, since it was very similar to binder A.

In Table 2.8, statistics of the averages between the various laboratories are provided, for all three binders. Reproducibility levels are not binder specific and

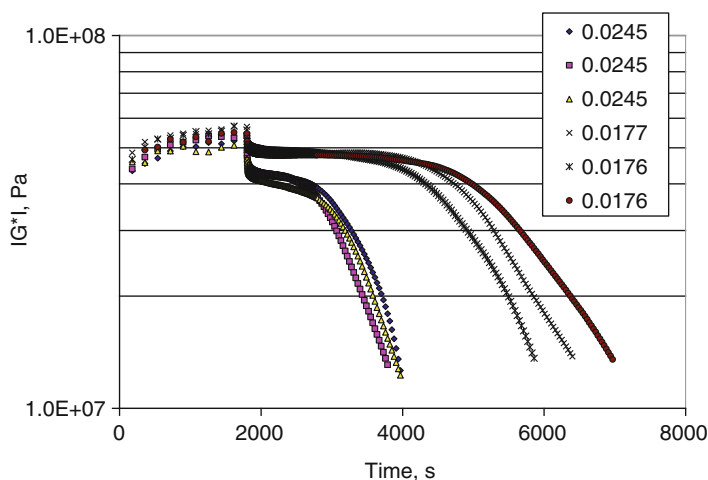


Fig. 2.48 Example of a typical fatigue curve at two strain levels with the three repetitions, (10°C, 10 Hz, Binder A)

are not reduced when going from the unmodified to the modified binder. Since the sample loading and equilibration times were fixed, the presence of physical hardening (binder A and binder C) does not influence the reproducibility. However, it should be stressed that if data from all the laboratories are used, the overall reproducibility is not very high, even though in literature a lot of reports have been published, demonstrating the various factors that can influence the results.

The coefficient of variation obtained in this study (when excluding two laboratories) is in agreement with the reproducibility level found in a recent round robin test (Eckmann et al. [20]). The reproducibility is slightly lower in comparison to levels found in a previous Rilem round robin test, where COV levels were below 10% (after excluding outliers) (Sybilski and Vanelstraete [49]).

2.2.4.2 Binder Fatigue Tests

A typical fatigue curve, including the equilibration period, is shown in Fig. 2.48, the fatigue strain levels are indicated. As expected, the modulus decreases, and this happens faster when strain is higher. When evaluating the fatigue data of the different participants some remarks can be made, in addition to the deviations seen in the low strain stiffness and phase angle data:

- one laboratory did not record the equilibration period
- for some tests (especially at high strains) the modulus dropped almost immediately when the fatigue test was started, these measurements were excluded from the analysis since this behavior is typical for an adhesive failure to one of the plates of the rheometer

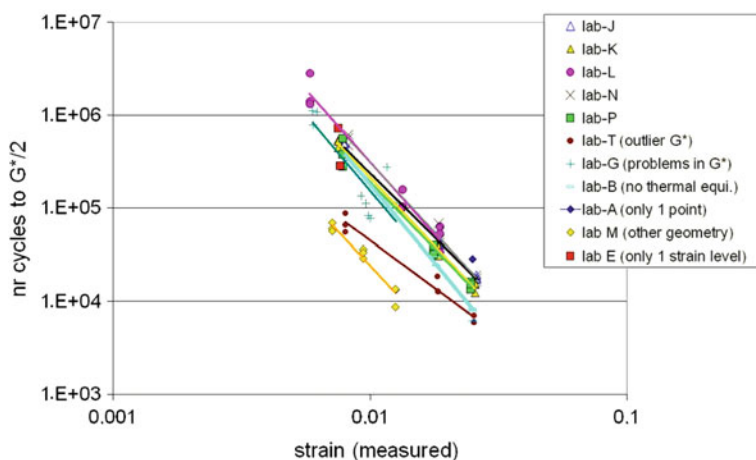


Fig. 2.49 Fatigue lines, based on a $\frac{1}{2} |G^*|$ low strain level for all data (Binder A)

- for lab G, the rheometer had problems recording the large number of data that are obtained in the fatigue test; at certain stages in the data, strain levels of zero, and modulus levels of $|G^*| > 10^{12}$ Pa were observed, these data were of course deleted from the analysis
- in general it was observed that the strain levels were not exactly the ones that were requested and they also tend to increase slightly with testing time.

The fatigue life was evaluated using the number of cycles needed to reduce the low strain level stiffness $|G^*|$ to half of its initial value. The results of this analysis for binder A, including all data, is given in Fig. 2.49. In order to compare the fatigue data it was necessary to recalculate the fatigue life at certain strain levels or the strain level at a certain life times, since each laboratory used slightly different strain levels. In Table 2.9, this statistical analysis is given, but only for laboratories without any deviations from the testing procedure. Data that were deviating in low strain stiffness levels were not included in the fatigue analysis. The data from lab M, which used the diabolo shaped geometry, were also not included in Table 2.9, since in Fig. 2.49 it seems that these data deviate from the ones measured with a DSR type equipment. In Table 2.9 the reproducibility levels seem to be rather good for this binder, as long as calculations are done within the measurement range.

Binders B and C were analyzed in the same way. Since binder B was very similar to binder A, only the outcome of the statistical analysis is given in Table 2.10. Again the reproducibility level of binder B seems to be rather good.

The fatigue lines of binder C are shown in Fig. 2.50, while the statistical analysis is given in Table 2.11. Binder C is quite different from the other two and its fatigue resistance is clearly higher; therefore most participants did not get a failure at the lowest strain level. The reproducibility for binder C is considerably reduced. At this stage it is not clear whether the reduced reproducibility is related to the nature of the binder (polymer modified) or is due to the longer testing time.

Table 2.9 Overview of trend lines based on the fatigue lines in Fig. 2.49, the strain levels for a certain fatigue life are calculated using the trend line (Binder A)

Trend line: log (lifetime) = log (intercept) + slope*log (strain)			Strain levels calculated from the trend lines for the lifetimes indicated in upper row:			
Binder A	Slope	Intercept	1.00E + 06	1.00E + 05	1.00E + 04	1.00E + 03
Lab-A	−2.7864	0.6273	5.9E-03	1.4E-02	3.1E-02	7.1E-02
Lab-J	−2.7474	0.7599	5.9E-03	1.4E-02	3.2E-02	7.3E-02
Lab-K	−2.9179	0.3111	5.9E-03	1.3E-02	2.9E-02	6.3E-02
Lab-L	−3.1081	0.1907	6.9E-03	1.4E-02	3.0E-02	6.4E-02
Lab-N	−2.9922	0.3215	6.8E-03	1.5E-02	3.2E-02	6.8E-02
Lab-P	−2.8588	0.3651	5.6E-03	1.3E-02	2.8E-02	6.3E-02
Average	−2.9018	0.4293	6.2E-03	1.4E-02	3.0E-02	6.7E-02
StDev	0.13	0.22	5.3E-04	8.1E-04	1.6E-03	4.5E-03
COV %	4.6		8.5	5.9	5.1	6.8

Table 2.10 Overview of average fatigue life calculated using trend lines, the laboratories that were included are similar as the ones used in Table 2.8 (Binder B)

Trendline: log (lifetime) = log (intercept) + slope*log (strain)			Strain levels calculated from the trend lines for the lifetimes indicated in upper row:			
Binder B	Slope	Intercept	1.00E + 06	1.00E + 05	1.00E + 04	1.00E + 03
Average	−2.9036	0.6213	6.8E-03	1.5E-02	3.3E-02	7.4E-02
StDev	0.20	0.39	7.8E-04	1.1E-03	1.9E-03	6.3E-03
COV %	6.9		11.4	7.0	5.6	8.5

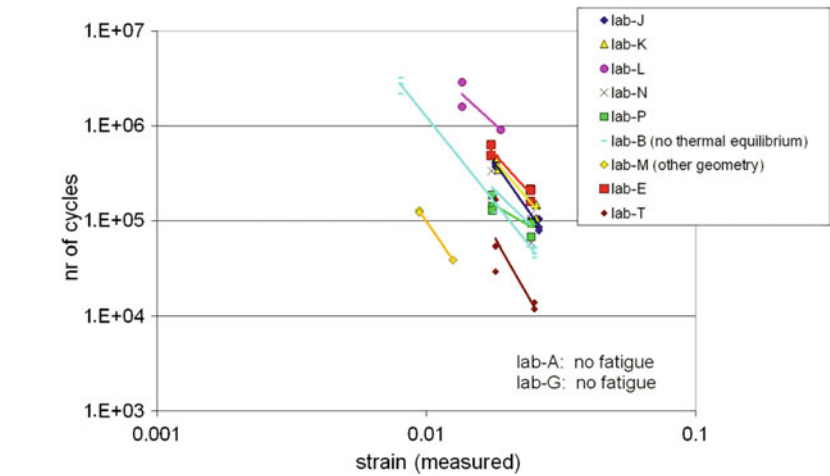


Fig. 2.50 Fatigue lines, based on IG^* low strain level $\frac{1}{2}$ for all data (binder C)

Table 2.11 Overview of trend lines based on the fatigue lines in Fig. 2.50, strain levels for a certain fatigue life are calculated using the trend line (Binder C)

Binder C	Trend line: $\log(\text{lifetime}) = \log(\text{intercept}) + \text{slope} * \log(\text{strain})$		Strain levels calculated from the trend lines for the lifetimes indicated in upper row:			
	Slope	intercept	1.00E + 06	1.00E + 05	1.00E + 04	1.00E + 03
Lab-A	-3.4411	0.3807	7.0E-03	1.4E-02	2.7E-02	5.2E-02
Lab-J	-4.0472	0.0346	8.1E-03	1.4E-02	2.5E-02	4.5E-02
Lab-K	-3.2995	0.7387	6.9E-03	1.4E-02	2.8E-02	5.6E-02
Lab-L	-2.5918	30.953	7.5E-03	1.8E-02	4.4E-02	1.1E-01
Lab-N	-2.7441	3.3735	4.4E-03	1.0E-02	2.3E-02	5.4E-02
Lab-P	-1.6936	160.25	1.5E-03	5.7E-03	2.2E-02	8.7E-02
Lab-B	-3.5520	0.0978	5.6E-03	1.1E-02	2.0E-02	3.9E-02
Lab-E	-3.1319	1.6886	6.9E-03	1.4E-02	3.0E-02	6.2E-02
Lab-T	-4.9847	0.0001	6.2E-03	9.9E-03	1.6E-02	2.5E-02
Average	-3.2762	21.9463	6.0E-03	1.2E-02	2.6E-02	5.9E-02
StDev	0.9283	52.8185	2.0E-03	3.6E-03	8.0E-03	2.5E-02
COV %	28.3		33.6	29.3	30.5	42.7

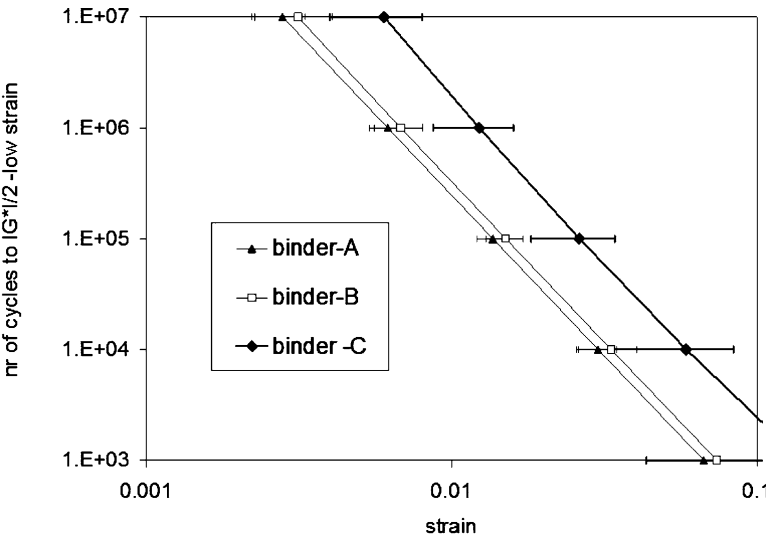


Fig. 2.51 Fatigue lines for the three binders based on average values using data from six laboratories, error bars are derived from the standard deviation

The average values, presented in Tables 2.9, 2.10 and 2.11, are plotted in Figs. 2.51 and 2.52. The error bars are derived from the standard deviations also shown in Tables 2.9, 2.10 and 2.11. Even though the error bars for binder C are large, it is still obvious that this binder is better in this fatigue test compared to the

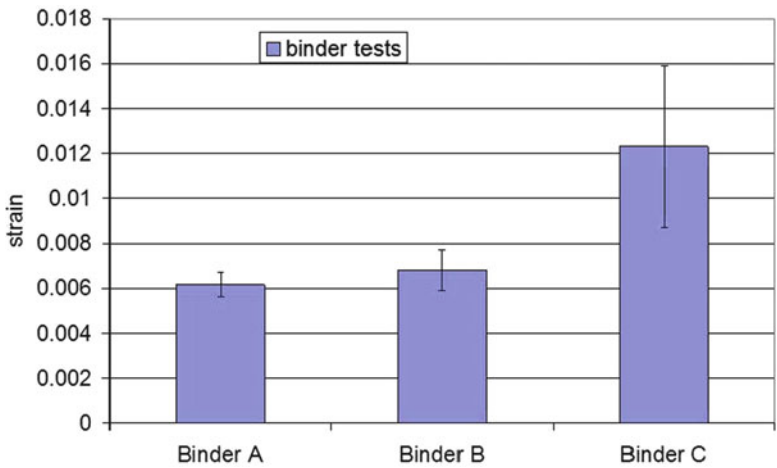


Fig. 2.52 Values of ϵ_6 with confidence intervals

Table 2.12 Overview of repeatability levels of the binder fatigue test (three repetitions, Binder A). Data in the last row related to 2 PB mixture fatigue data

COV (%)	High strain	Intermediate strain	Low strain
Lab-J	5.5	7.5	16.9
Lab-K	12.3	12.4	10.0
Lab-L	29.4	27.2	45.9
Lab-N	7.2	27.4	12.8
Lab-P	8.7	10.8	35.3
Mixture fatigue 2 PB (six repetitions)	20	14	19

other two binders. The behavior of binders A and B with regard to fatigue testing can be considered as identical.

Finally the repeatability levels were also evaluated and some results are shown for binder A, in Table 2.12. Table 2.12 shows that the repeatability levels are not very high, and tend to reduce if the fatigue life becomes longer. But they are still in the range of levels obtained in the mixture fatigue test, also indicated in Table 2.12. Lab-L measured at lower strain levels compared to other participants, so this could explain why the values for this lab are different. The repeatability levels in binder tests are based on only three repetitions, and since the levels are worse than the reproducibility levels, it may indicate that three repetitions are not enough.

2.2.4.3 Mixture Fatigue Tests

Fatigue test results on the MA, MB and MC mixes are depicted in Fig. 2.53. Mix MC, whatever the procedure used (2 or 4 PB) shows clearly better behavior in fatigue. Table 2.13 gives the fatigue criteria ϵ_6 according to the mixes and the

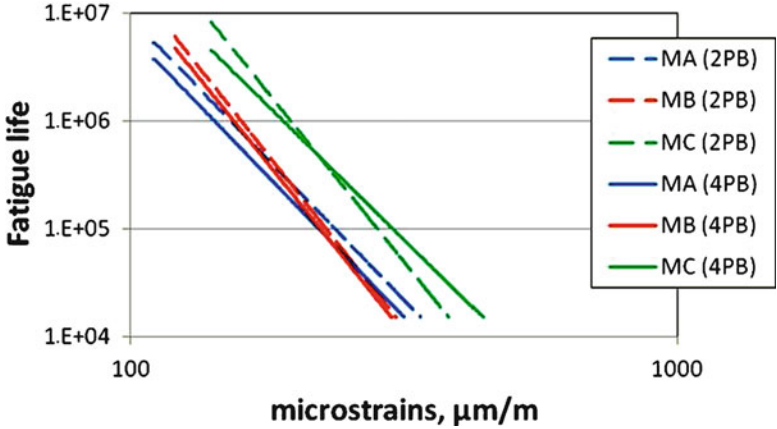


Fig. 2.53 Fatigue lines for the three mixes MA, MB and MC using the two fatigue test (2 pts: two points bending, 4 pts: four point bending)

Table 2.13 Outcome of the two mixture fatigue tests

Mix	Epsilon 6–2 PB	Slope – 2 PB	Epsilon 6–4 PB	Slope – 4 PB
MA	152(min 149, max 155)	-5.23 ± 0.03	142(min 134, max 151)	-5.25 ± 0.78
MB	159(min 152, max 166)	-6.43 ± 1.10	154(min 150, max 158)	-5.73 ± 0.53
MC	196(min 187, max 207)	-6.3 ± 0.95	190(min 174, max 208)	-4.99 ± 1.37

fatigue procedure used. Mix MC has an ϵ_6 20% higher than the others. Mix MB seems to be slightly better than the mix MA, as was also found for binders (Fig. 2.52). Nevertheless, the confidence intervals are too large to definitely rank these two mixes. It is interesting to note that the ϵ_6 given by 2 and 4 PB fatigue tests are similar whereas the slope of the fatigue line seems to be “procedure dependent” (Fig. 2.54). It should be noted that this is in fact the second comparison tests and that the results obtained previously by LCPC (2 PB) and IBDM (4 PB) were also very close (Di Benedetto et al. [19]).

2.2.5 Second Phase Results

2.2.5.1 Binders: Low Strain Stiffness and Phase Angle

Similarly to Phase 1, it was requested that tests were performed at three strain levels with three repetitions. Figures 2.55 and 2.56 present the evolution of $|G^*|$ and phase angle in the equilibration period for sample D (one repetition), and Figs. 2.57 and 2.58 for sample F (the hardest one).

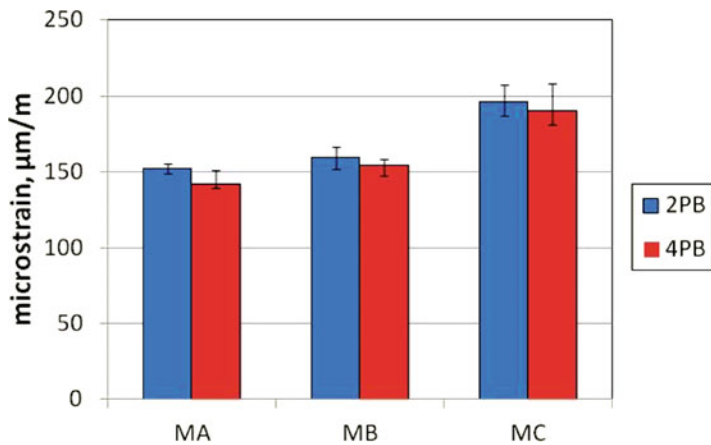


Fig. 2.54 Values of ε_6 with confidence intervals for the mixes

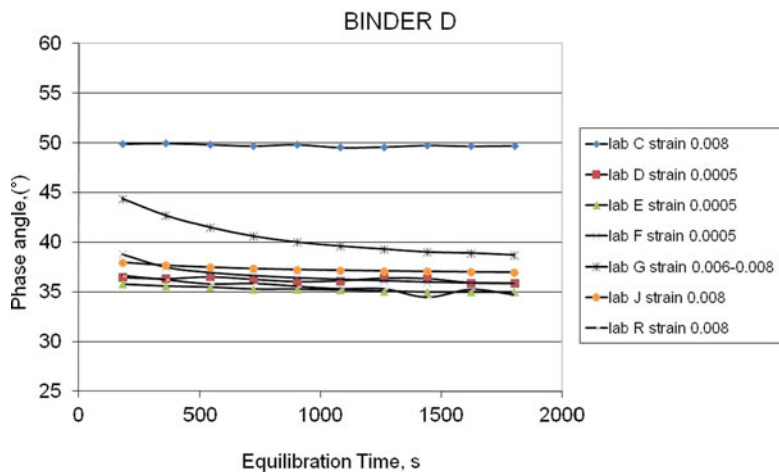


Fig. 2.55 Evolution of the phase angle during the equilibration period (Binder D)

Results from four laboratories C, D, G and M were estimated to be outliers (Table 2.16). Labs C and D: both labs used the same equipment DSR Haake MARS II. Even if the lesson learned from Phase 1 forced Lab D (T in Phase 1) to improve the temperature control of the sample providing the self-made temperature control chamber, the problem still existed. Results in Phase 2 from Lab D were obviously better than from Lab C (the same type of DSR), for instance G^* higher by 65%, and closer to average results from other laboratories using other types of DSRs.

Another specific case was Lab M which solved the problem of delamination of the sample from DSR steel plates identified in Phase 1. G^* results reported from this

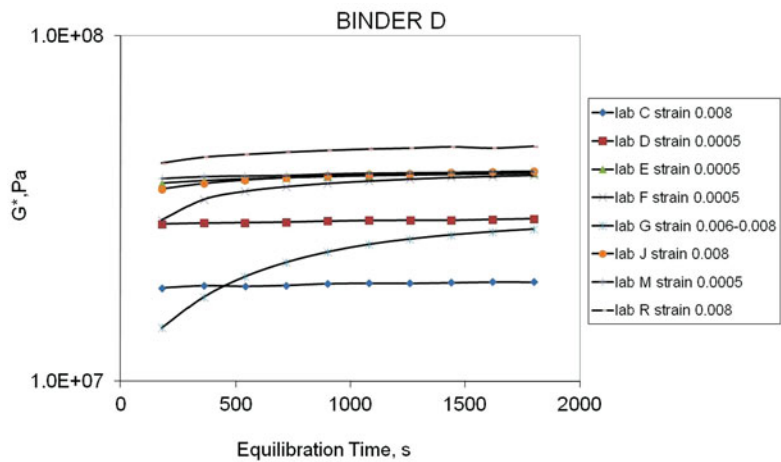


Fig. 2.56 Evolution of the complex modulus norm in the equilibration period (Binder D)

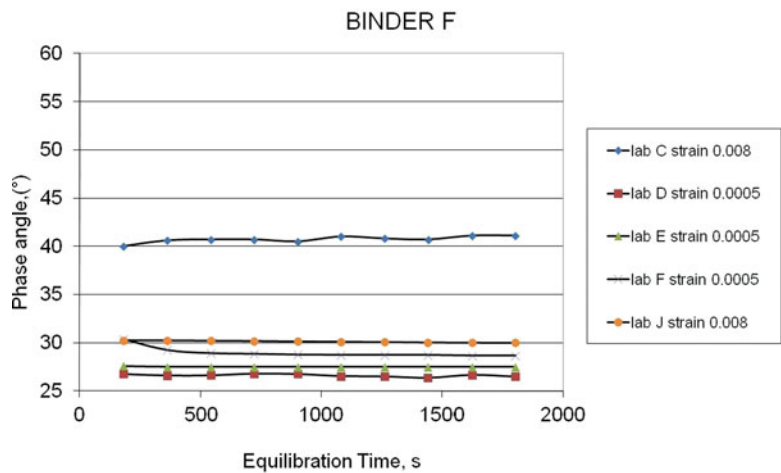


Fig. 2.57 Evolution of the phase angle in the equilibration period (Binder F)

lab in Phase 2 were accepted, but results of phase angle were excluded. The reason of erroneous results could not be found.

The most difficult problem in Phase 2 occurred in testing binder F, the most stiff (Pen 10/20) among the binders tested. Only six labs delivered their results, and many laboratories reported problems testing this hard binder. Obviously, the test temperature 10°C is too low for this grade. A proposal of the solution is described further.

Tables 2.14 and 2.15 present the statistics of tests of all laboratories provided for binders D and F, respectively. Average values of G^* and phase angle are presented.

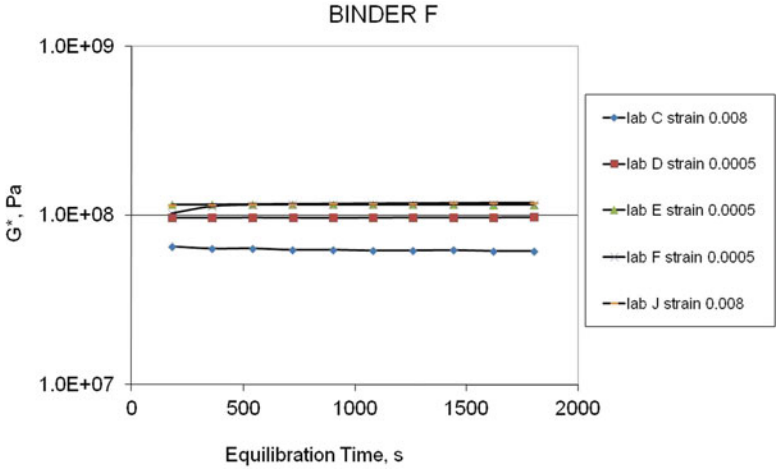


Fig. 2.58 Evolution of the complex modulus norm in the equilibration period (Binder F)

Table 2.14 Overview of average values of low strain $|G^*|$ and low strain phase angle (10 Hz, 10°C, Binder D)

Lab	Av. $ G^* $ (Pa)	cov% G (%)	Av. δ (°)	cov% δ (%)
C	1.88E + 07	3.0	50.2	0.9
D	3.11E + 07	5.3	35.7	1.3
E	4.17E + 07	5.2	34.9	0.5
F	3.91E + 07	3.7	35.9	0.7
G	2.79E + 07	3.1	38.5	0.4
J	4.14E + 07	5.4	36.9	0.6
M	4.26E + 07	5.3	0.0 (?)	26.3
R	4.51E + 07	5.3	35.1	1.3

Table 2.15 Overview of average values of low strain $|G^*|$ and low strain phase angle (10 Hz, 10°C, Binder F)

Lab	Av. $ G^* $ (Pa)	cov% G (%)	Av. δ (°)	cov% δ (%)
C	6.06E + 07	14.2	40.7	4.4
D	9.76E + 07	5.0	26.6	1.4
E	1.18E + 08	2.1	27.3	0.5
F	1.17E + 08	2.4	28.5	0.9
J	1.21E + 08	2.4	29.9	0.5
R	1.12E + 08	0.0	26.8	1.8

Coefficient of variation was calculated as the average value divided by the standard deviation, (multiplied by 100, if in percentage).

Table 2.16 presents the statistics of average values between the various laboratories, for all three binders. Reproducibility levels are not very high not

Table 2.16 Overview of average values of low strain |G*| and low strain phase angle data, for all binders, Phase 2

Binder	All data		Excluding data from lab		Excluding data from lab	
	G* (Pa)	δ (°)	G* (Pa)	δ (°)	G* (Pa)	δ (°)
D	All data		Excluding data from lab C, M (phase)		Excluding data from lab C, D, G, M (phase)	
Average	3.60E + 07	33.4	3.84E + 07	36.2	4.20E + 07	35.7
StDev	9.12E + 06	14.4	6.41E + 06	1.3	2.18E + 06	0.9
COV %	25.3	43.1	16.7	3.7	5.2	2.5
E	All data		Excluding data from lab C, M (phase)		Excluding data from lab C, D, G, M (phase)	
Average	4.89E + 07	29.4	5.17E + 07	31.7	5.62E + 07	31.2
StDev	1.12E + 07	12.2	8.39E + 06	1.2	3.78E + 06	0.6
COV %	22.9	41.4	16.2	3.7	6.7	1.9
F	All data		Excluding data from lab C		Excluding data from lab C, D	
Average	1.04E + 08	30.0	1.13E + 08	27.8	1.17E + 08	28.1
StDev	2.30E + 07	5.4	3.71E + 06	13.7	3.71E + 06	1.4
COV %	22.0	18.1	3.2	4.9	3.2	4.8

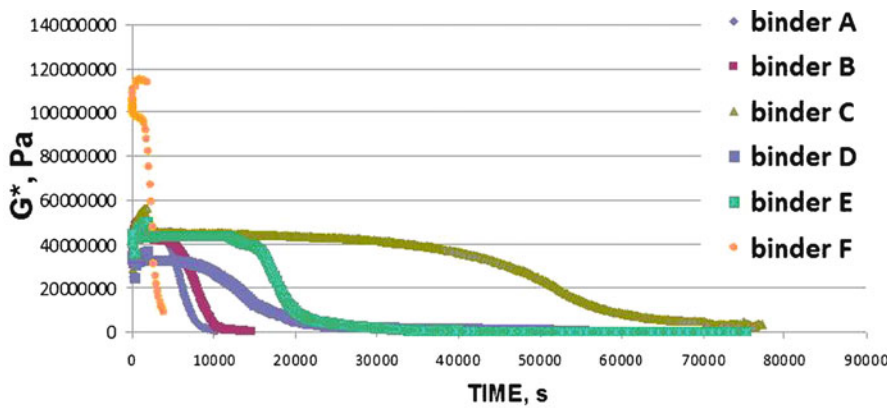


Fig. 2.59 Typical fatigue curves for all binders (Phase 1 and 2)

excluding outliers. After excluding outliers, the coefficient of variation is below 10%, in agreement with results of the first phase test and literature.

2.2.5.2 Binder Fatigue Tests

Figure 2.59 presents the typical fatigue curves for all binders tested in Phase 1 and 2. It is noticed that the hardest binder F shows the highest G* value and the shortest fatigue life. Further, from the group of binders in Fig. 2.59 it may be noticed that PmB binder C shows different behavior from the straight run bitumens, and the

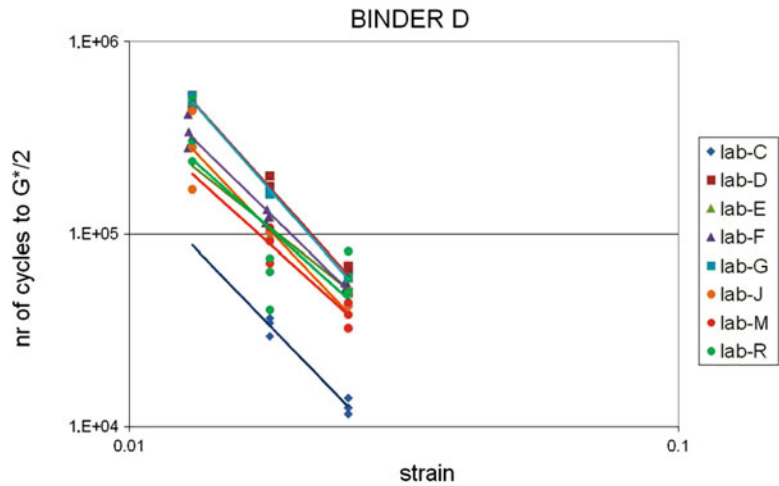


Fig. 2.60 Fatigue lines, based on $|G^*|$ low strain level/2 for all data (Binder D)

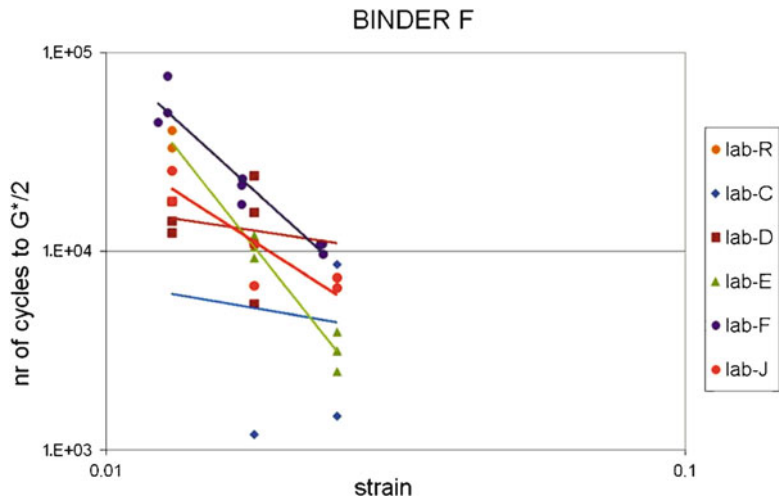


Fig. 2.61 Fatigue lines, based on $|G^*|$ low strain level/2 for all data (Binder F)

longest fatigue life. This is confirmed in Figs. 2.60 and 2.61, comparing results of fatigue test on binders D and F.

Figure 2.60 shows the results of calculation of fatigue life presented as a number of load cycles to obtain the half of initial G^* value for binder D. As described before, the results from Lab C are obvious outliers, excluded from further analysis. In this case, the reason lies in insufficient temperature control of the DSR used (Haake). Tests on binder E (Pen 20/30) showed similar results.

Table 2.17 Overview of trend fatigue lines, the strain levels for a certain fatigue life are calculated using the trend line (all binders Phase 2)

	Trend line: $\log$ (lifetime) = $\log$ (intercept) + slope $\cdot\log$ (strain)		Strain levels calculated from the trend lines for the lifetimes indicated in upper row:			
	Slope	Intercept	1.00E + 06	1.00E + 05	1.00E + 04	1.00E + 03
Binder D						
Average	-2.95	1.42	0.0091	0.0198	0.0435	0.0959
StDev	0.29	1.46	0.0013	0.0015	0.0024	0.0115
COV%	-9.8	102.7	14.3	7.5	5.6	12.0
Binder E						
Average	-3.57	0.16	0.0096	0.0184	0.0353	0.0680
StDev	0.34	0.27	0.0013	0.0019	0.0043	0.0118
COV%	-9.5	170.8	13.1	10.5	12.1	17.4
Binder F						
Average	-2.26	3.24	0.0029	0.0077	0.0213	0.0608
StDev	0.54	3.88	0.0018	0.0030	0.0031	0.0064
COV%	-24.0	119.7	60.8	38.8	14.6	10.5

Figure 2.61 presents results for binder F. It is easily seen that this binder was the most difficult to test. Only six laboratories reported results. These results show a large scatter. Some of the laboratories could not obtain results at the lowest strain (0.013) requested. In the case of binder F the testing temperature 10°C is obviously too low – the stiffness of binder is too high.

The fatigue trend lines with the repeatability levels evaluated in the second phase are presented in Table 2.17, and on Fig. 2.62. The conclusions are similar to the first phase. The lowest repeatability has been achieved for binder F (the hardest one). The repeatability levels are based on three repetitions, and the number of accepted results is relatively lower (in case of binder F even two labs only).

Comparison of fatigue life evaluation of all binders tested in Phases 1 and 2 may be found on Fig. 2.63 presenting ε_6 values from the binder fatigue tests. The highest ε_6 value was obtained for binder C (PmB), and the lowest for the hardest binder F as expected. These results confirm the applicability of the binder fatigue test. This conclusion has been further supported by results of mixture fatigue tests and comparison of binders and mixture fatigue tests results.

2.2.5.3 Proposal for Testing Harder Binder Grades

Binder F of harder grade (10/20) than other binders tested in this Round Robin Test caused more problems in testing at the chosen temperature 10°C. The stiffness of the binder at this temperature was too high, and this was the reason for the problems in the majority of laboratories. Too stiff a binder sample leads to the unacceptable damage of the material during test (Fig. 2.64).

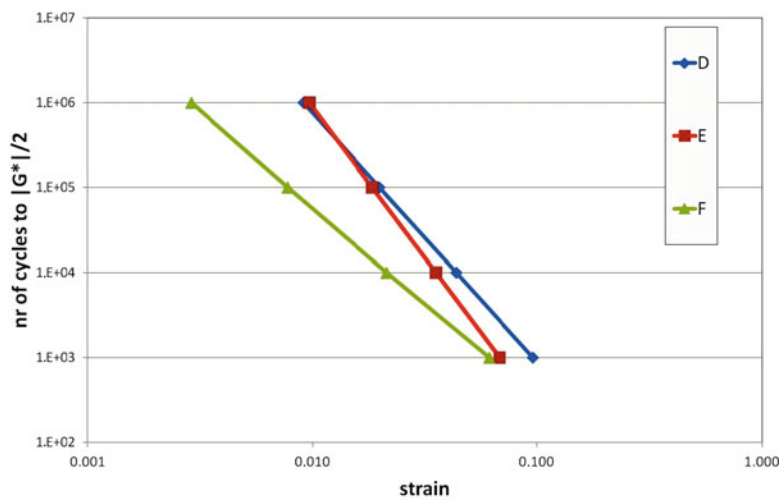


Fig. 2.62 Fatigue lines for three binders based on average values using data from laboratories in second phase

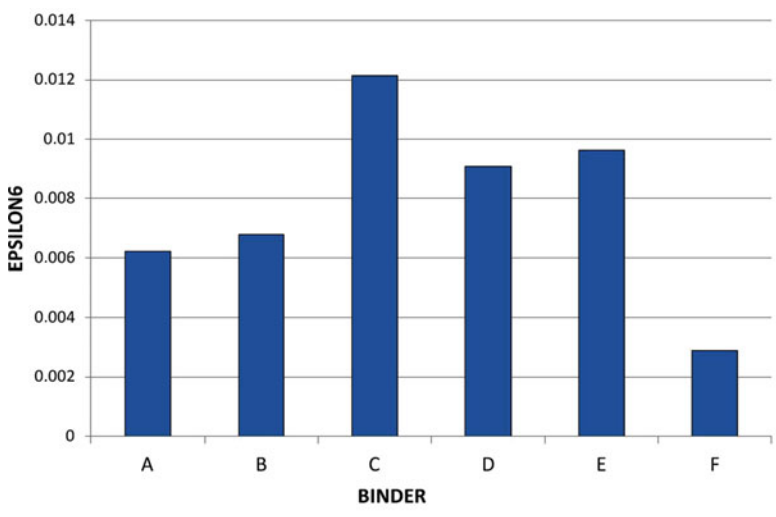


Fig. 2.63 Binder fatigue life ϵ_6 of all binders (Phase 1 and 2)

The proposed solution is to find an equivalent temperature for fatigue test in case of very hard binders. The binders used in RRRBFT which were successfully tested (A, B, C, D, E) exhibited norm of shear modulus obtained in oscillatory tests for frequency $f = 10$ Hz at $T = 10^\circ\text{C}$ of a significantly lower level than binder F. This is presented in Table 2.18. The results presented were obtained at a strain in most cases close to $\gamma = 0.0005$. The values presented are the average of results from RRRBFT after exclusion of outliers.



Fig. 2.64 Example of problems with fatigue test: the damage of the sample – binder F

Table 2.18 Average value of G^* of binders tested in RRRBFT (1 and 2 phase)

Binder	A	B	C	D	E	F
Norm of shear modulus, Pa	4.83E + 07	4.94E + 07	5.55E + 07	4.20E + 07	5.62E + 07	1.17E + 08
Average phase angle, °	37.3	38.4	33.3	28.6	25.2	28.1

To reduce the stiffness of binder F the test of shear modulus was performed at temperature 20°C (Fig. 2.65). It may be seen that maximum value of complex shear modulus for binder F (10/20) at 20°C is in the range of values obtained for softer binders at 10°C.

The range of values of G^* of five softer binders tested at 10°C is from 42.0 to 56.2 MPa, and the average value of G^* is 50.28 MPa. The value of G^* of binder F at 20°C is 45.0 MPa. It means that G^* of binder F at 20°C is in the range of other binders at 10°C. It may be assumed that the fatigue test of stiffer binder may be performed at higher temperature at which the equivalent shear modulus G^* of binder is achieved. This temperature is the equivalent temperature of the same value of shear modulus – equi-stiffness temperature (EST).

This proposal seems to be a proper solution for testing the binder fatigue of stiffer bitumen grades than 20 Pen.

Besides the change of test temperature for harder binder grades, it is recommended to reduce the fatigue test strain level. The difference is shown in Figs. 2.66 and 2.67. It may be observed that in case of higher strain level (0.013 at 20°C) is not possible to carry out binder fatigue tests. Instant breaking of the sample may be observed which occurs in the sample or between sample and rotor.

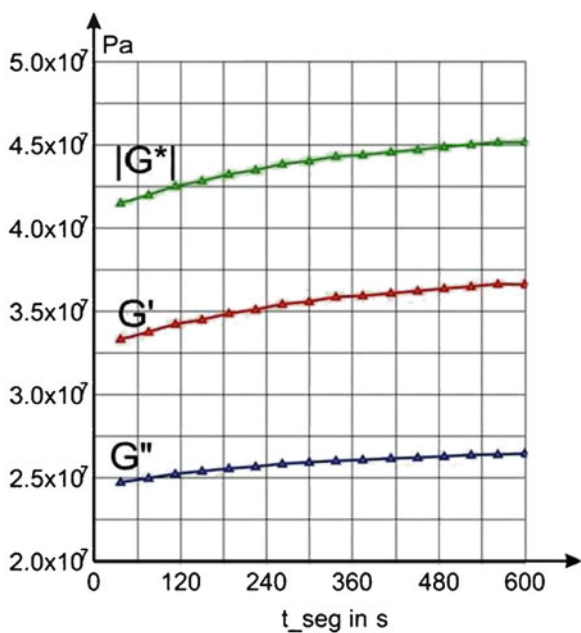


Fig. 2.65 Norm of complex shear modulus as a function of time (checking for thermal equilibrium) – additionally the real and imaginary parts of complex shear modulus

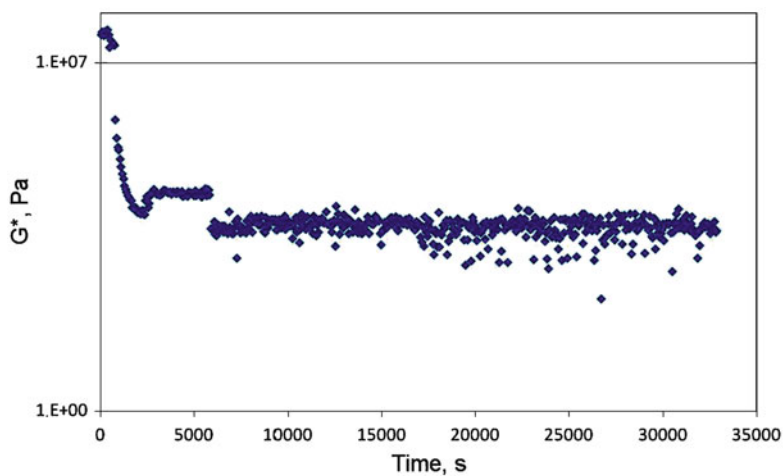


Fig. 2.66 Norm of complex shear modulus as a function of time for shear strain $\gamma = 0.013$ at 20°C

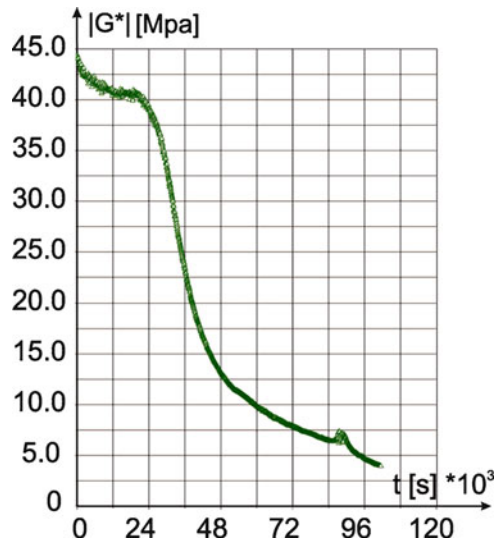


Fig. 2.67 Norm of complex shear modulus as a function of time for shear strain $\gamma = 0.008$

Table 2.19 Outcome of the two mixture fatigue tests (average values)

Mix	Epsilon 6–2 PB	Slope – 2 PB	Epsilon 6–4 PB	Slope – 4 PB
MD	191	–5.44	197	–5.85
ME	188	–6.62	200	–5.50
MF	151	–4.98	174	–6.20

Similar effect can be observed for shear strain $\gamma = 0.018$ and $\gamma = 0.025$. It is postulated that the test should be performed at lower strain level (e.g. 0.008).

2.2.5.4 Mixture Fatigue Tests

The testing was performed with two methods LCPC – 2 PB (10°C, 25 Hz) and IBDM – 4 PB (10°C, 10 Hz) as in the first phase. Table 2.19 presents the results of MD, ME, MF mixtures. Similarly to previous tests, the same agreement between both methods is observed. Comparison is shown on Fig. 2.68.

Fatigue criteria ϵ_6 and slope of the regression line are presented in Table 2.19. Mixture MF, with the hardest binder F, exhibits the shortest fatigue life in comparison to mixtures MD, and ME with softer binders D, and E.

Similarly as in Phase 1, the results of the mixtures fatigue test by 2 and 4 PB in Phase 2 provided values of the ϵ_6 which were relatively close. The slope of the fatigue lines differ as in the Phase 1, depending on the test procedure.

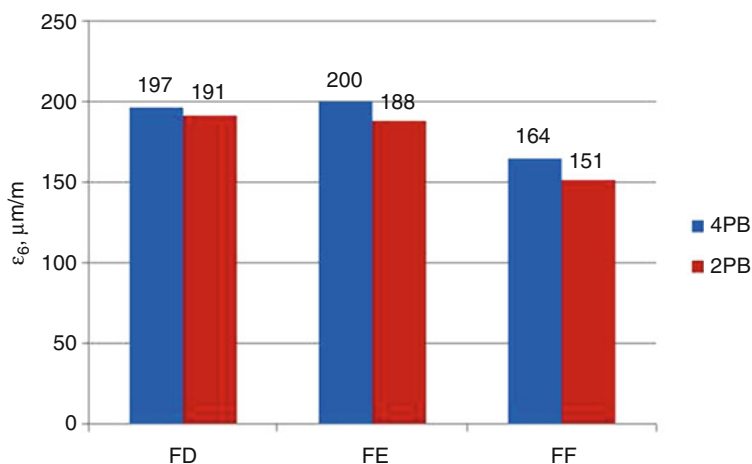


Fig. 2.68 Comparison of mixture fatigue tests RRRBFT second phase

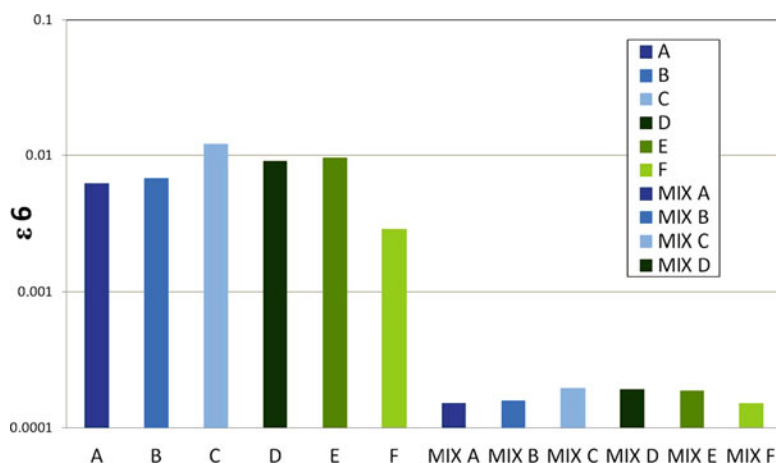


Fig. 2.69 Comparison of binders and mixes fatigue life expressed as ϵ_6 (Phase 1 and 2), data from 2 PB test

2.2.5.5 Comparison of Binder and Mixture Fatigue Test Results

Two parameters were used for evaluation of fatigue life of both materials (binders and asphalt mixtures) – the slope of fatigue line and ϵ_6 . Comparison of both parameters for both materials is presented in Figs. 2.69 and 2.70.

In both cases a good agreement may be seen between results of binders or mixtures fatigue tests. Of course, the values of both parameters are different but the ranking of the materials is the same, which is the most important conclusion. The binder fatigue test is a reasonable method for estimating the fatigue life of

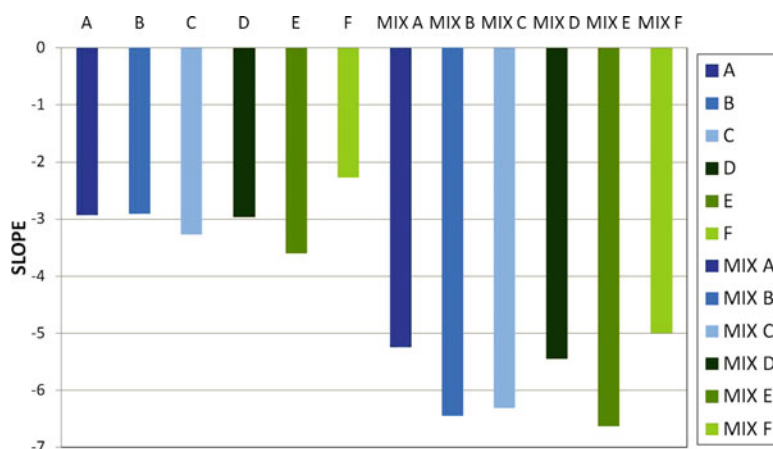


Fig. 2.70 Comparison of binders and mixes fatigue life expressed as fatigue line slope (Phase 1 and 2), data from 2 PB test

asphalt mixtures. This provides a much less time consuming and less costly laboratory test method for choosing proper binder in the first stage of asphalt mixture design and testing.

2.2.6 Discussion and Conclusions

The first step of the evaluation of the binder fatigue test results was the review of the low strain modulus and the phase angle in the equilibration phase.

In the first phase of test, results from 11 labs were analysed. For the low strain modulus and phase angle, at least two, or even three data sets, needed to be excluded to get reproducibility levels common for bitumen stiffness data. After elimination of three outliers the Coefficient of Variation (COV) improved for G^* for binder A to 11.6%, B to 15.1% and C to 15.6%, and for phase angle A to 3.7%, B to 2.0% and C to 3.5%.

The number of outliers means that about 20% of testing equipments has problems. In this group of participants one problem could be related to a very slow equilibration, and another problem could be linked to an insufficient temperature control of the sample between the plates of the rheometer.

The conclusions from the first phase binder fatigue test are the following:

- The results of binder fatigue tests of straight run bitumen (binders A and B) present a reasonable reproducibility, after eliminating outliers. Results of six participants were included in the analysis. The repeatability levels for binder A and B were less good compared to the reproducibility levels – this is a strange finding and may indicate that three repetitions per strain level are not enough, for instance in the mixture fatigue tests the common number of six samples is used per strain level. The results of binder C (PmB) are of low repeatability and

reproducibility, but the fatigue life of this binder was much higher, and it seems to be more difficult to measure longer fatigue life. The geometry also seems to have an effect on the result, but up to now only two geometries were used, and one only in one lab.

- In the second phase results from eight laboratories were analysed. The observation from the first phase made one of the laboratories to improve the temperature control in the binder sample. It improved the results of testing, but not to the expected level. The basic incorrectness of the DSR used could not be eliminated.
- For the low strain modulus and phase angle two or four data sets needed to be excluded. After elimination of four outliers the coefficient of variation improved for G^* for binder D to 5.2%, E to 6.7% and F to 3.2%, and for phase angle D to 2.5%, E to 1.9% and F to 4.8%. Outliers posed 25–50% of participating laboratories.

The conclusions from the second phase binder fatigue test are following.

- Binders used in the second phase represented straight run bitumen of various grades and various consistencies. Fatigue tests results of softer binders D and E present a reasonable reproducibility, after eliminating outliers. The most problems were exhibited by the testing of the hardest binder F.
- The ranking achieved in binder fatigue tests is similar in comparison to mixture fatigue tests for all binders. It is one of the most valuable conclusions considering the variability of binders tested – grade of straight run bitumen from 10/20 to 50/70, and type of binder: unmodified or polymer-modified. The results prove the applicability of binder fatigue test as a preliminary evaluation of the bituminous mixture fatigue test.

The main conclusions from the binder fatigue test are following:

- The binder fatigue test is an important tool for preliminary evaluation of the mixture fatigue.
- The binder fatigue test procedure should consider the consistency (grade) of binder tested – in case of harder grades (e.g. 10/20 Pen) higher temperature, and lower strain should be applied.
- The Dynamic Shear Rheometer (DSR) may be effectively used for binder fatigue tests.
- The DSR used for binder fatigue tests has to be equipped with a temperature control system capable of providing a proper control of temperature in the binder sample; the isolating properties of bitumen has to be considered – it has been proven that temperature control system in only the bottom steel plate only is not effective – both steel plates have to be equipped in temperature control system.
- To control the condition of the rheometer, it is recommended to participate in regular round robin tests, which seem to be the best way to detect possible deviations in the testing device.
- The above findings are also a message to the producers of rheometers, very often a reliable temperature calibration method is not provided, and a yearly recalibration of the temperature is often not included in their maintenance contracts.
- A binder fatigue test procedure for DSR has been developed.

2.3 Test Description: RILEM Binder Fatigue Test

2.3.1 Preparation of the Rheometer

The procedure has been developed for Dynamic Shear Rheometer Paar Physica MCR 500. For other types of DSR some adaptations may be needed or useful.

- Binder fatigue testing is performed with the 8 mm plates, and a standard gap setting of 2 mm (in some cases exceptions may be applied).
- The zero gap is set at the fatigue test temperature. In this test protocol a temperature of 10°C was selected.

2.3.2 Sample Preparation and Loading

- For the 8 mm plates diameter ($R_0 = 4$ mm, Fig. 2.74), it is advisable to prepare samples in silicon molds (these molds are somewhat larger than the dimensions needed in the rheometer, the diameter of the molds is 8.5 mm, and the height is 2.5 mm)
- First, the sample is homogenized in an oven. For the unmodified binders, an oven temperature of 160°C is sufficient. For the PmB sample, an oven temperature of 180°C is recommended. The heating time is restricted, depending on the sample size in order to prevent ageing – for half liter cans 1.5 h is sufficient; for a 100 ml penetration cup 1/2 h is sufficient. Samples should be stirred before pouring into a mold (stirring may be done manually).
- The as-received sample should be homogenized, and further divided into ten subsamples. For each subsequent test, a new subsample should be homogenized.
- After the sample has been homogenized, it is poured in the mold. Molds should be left at room temperature (from 18 to 24°C) to cool the sample.
- The sample is loaded within 60 min after molding into the rheometer. The temperature of the rheometer plates during loading should be high enough in order to assure a good adhesion to the plates. In case of unmodified binders, 60°C temperature is recommended. In any case, the sample should start flowing to get good adhesion to both plates. The gap is first decreased to $h = 2.05$ mm. This operation should be completed during 10 min. If a hold function is available to measure normal vertical stress, it should be checked.
- The sample is cooled to the test temperature (10°C recommended in this project) at a rate of 2°C/min. Then it is trimmed using a heated spatula. Afterwards the gap is decreased to exactly $h = 2.0$ mm.
- Before starting the fatigue tests, the sample should achieve the correct temperature, and this period is referred to as the equilibration period. An equilibration period of 30 min is recommended at the test temperature of 10°C. During the equilibration period the sample modulus can be tested at a very low strain level



Fig. 2.71 Mixing of the pre-polymer and catalyst

at every 3 min (at least ten measuring points should be obtained before starting the fatigue test). The equilibration period may be prolonged up to maximum 1 h. A test frequency during the equilibration period of 10 Hz is recommended.

- Report the thermal history during sample preparation, and the modulus and phase angle tested during the equilibration period at 10 Hz, and tested at the lowest available strain level.

2.3.3 Preparation of Silicon Molds

- The molds for DSR testing, are prepared from a two component system, a silicon pre-polymer is mixed with a catalyst, this (viscous) solution is brought in the desired shape and either left 1 day at room temperature or heated to 60°C then the reaction is complete in about 1 h. The pre-polymer and catalyst should be mixed in a ratio of 100 pre-polymer to 10 of the catalyst (10% of catalyst in the mixture). This proportion is based on experience with use of product of Kemitura (pre-polymer RTV 896 A, and catalyst RTV 896 B). It can vary depending on the producer.
- Silicon mold preparation is shown in Figs. [2.71](#), [2.72](#) and [2.73](#).



Fig. 2.72 Before making of molds, the shape for silicon rubber is needed – the metal pieces are recommended

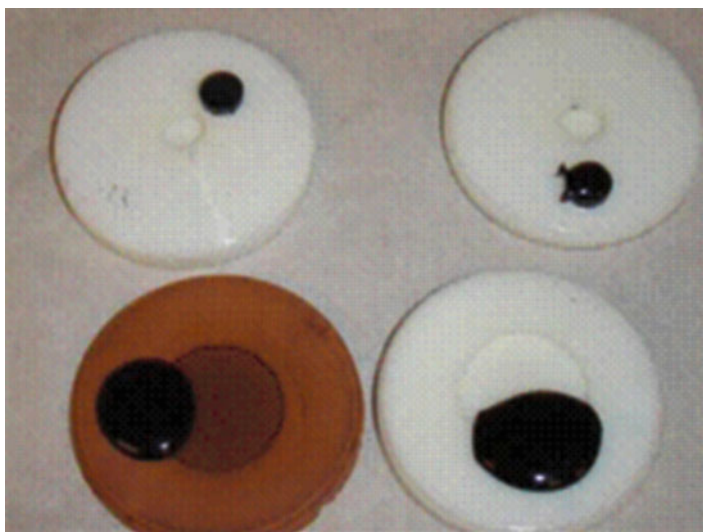
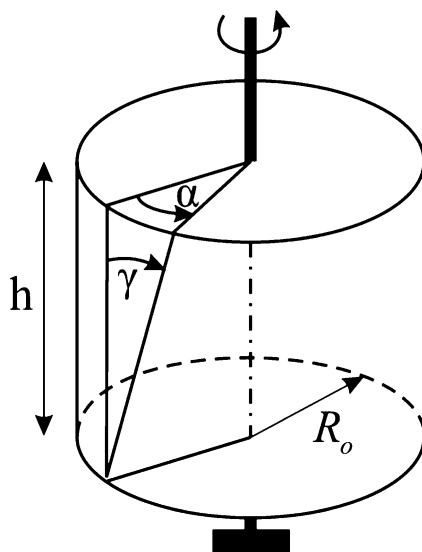


Fig. 2.73 The silicon rubber molds, and bitumen samples

2.3.4 The Fatigue Test: Testing Mode

- Required (preferred) testing mode is strain controlled mode. However, if a laboratory can test in stress controlled, it is encouraged to test in both modes (due to practical reason, if the sample “breaks” in the controlled stress mode, and the test is not stopped, this could harm the rheometer, if this happens in controlled strain mode, the applied stress decreases in accordance, and there is no risk to the rheometer).

Fig. 2.74 Schematic view of the sample, shear strain angle interpretation



- The frequency: a high frequency, typically 10 Hz, is recommended – because then for the same testing time the number of load cycles is larger.
- The strain levels: It is recommended to perform the test at three levels of strain (see Fig. 2.74), the highest level of strain is $\gamma = 0.015\text{--}0.025$, the intermediate strain level is $\gamma = 0.01\text{--}0.013$, the lowest level is $\gamma = 0.004\text{--}0.009$. The time for one test should not exceed 1.5 days.
- It is recommended that the fatigue test is performed until the binder stiffness modulus G^* reaches 20% of the initial value. Due to differences of fatigue behavior of straight run bitumen and PmB it is recommended to gather test results allowing for the analysis of the results with a conventional as well as dissipated energy approach.
- Three replicates at each strain level.
- Testing temperature: It is recommended to perform the test at a temperature at which the binder's $G^*\sin\delta$ at 10 Hz is 30–60 MPa (it is believed that at lower stiffness levels the edge effect may be observed, and at higher stiffness levels the equipment's compliance errors). A test temperature of 10°C is recommended for binders of the penetration range 20–70 dmm, and 20°C for binders of penetration below 20 dmm.
- After performing the fatigue test, it is strongly recommended to check the bottom and the upper plate, if the test is performed correctly, both plates should be covered with bitumen when the fatigue test is finished. If this is not the case, the adhesion between the plate and the binder was probably not sufficient, and the test needs to be repeated, and the temperature at which the binder is loaded into the rheometer can be increased. Figure 2.75 shows a good adhesion between the plates and the binder.



Fig. 2.75 Checking the adhesion between sample and plates after test

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Chapter 3

Mixture Design and Compaction

Hussain U. Bahia, Aaron Coenen, and Nader Tabatabaee

Abstract The majority of current bituminous mix design procedures depend heavily on volumetric analysis, and on measuring density after a given compaction effort. There have been many questions about the influence of the compaction method on performance, and the lack of suitable simulation of field compaction in the lab. There are also concerns about the interactive effects of the laboratory mixing and compaction temperatures on the laboratory compaction methods. While there are no agreements on which method is the best in terms of simulating field conditions, there is agreement that density alone does not correctly reflect the potential of mixture to resist the damages caused by traffic loading. One of the evolving concepts to complement density and more accurately estimate this potential is characterization of aggregate skeleton and particle to particle interaction using imaging technology. However, methods of reducing images to engineering characteristics that can complement mix design procedures, and perhaps allow better simulation of field compaction, are still not in reach. This section is focused on giving background about various widely-used compaction methods, and about imaging techniques used for characterizing bituminous mixtures. It also covers a project carried out by the RILEM group on introducing the newly developed iPas software for image processing and analysis of 2D images of asphalt mixtures. As a part of this work, the software was used and verified in a large experiment that included many leading bituminous testing laboratories, from various countries.

Keywords Bituminous mix design • Image processing • Aggregate orientation • Aggregate packing • Compaction

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3.1 Background

3.1.1 *Motivation for Study*

Bituminous mixtures have been utilized for pavement applications for over 150 years. Although early mixtures proved successful as paving materials, they were not designed with the same knowledge and understanding of the materials as today. Taking pavement performance criteria into account was recognized as a necessity among roadway engineers as the knowledge of paving materials grew. With an increasing knowledge came increasing expectations. This motivated the development of laboratory mix design and compaction to accommodate existing pavement conditions into the pavement design process, and allow for the design to best fit the intended pavement application (Gartner [13]).

Today, engineers design asphalt mixtures as a function of cost, safety, and performance. Asphalt-aggregate mixture and compaction methods are selected based on the influence of mixture components on the performance characteristics and volumetric properties of the paving mixture. Eight mixture properties have been considered as targets for optimization in a mix design method: stiffness, stability, durability, fatigue resistance, fracture characteristics, skid resistance, permeability, and workability (Monismith [26]). It is recognized that a sustainable balance of these properties should be achieved when selecting mixture components for a specific paving application.

While there is general agreement on the targets of mix design, modern design methods differ in their methods of compaction to prepare mixture samples in the laboratory. The challenge is to use a compaction method in the laboratory that accurately simulates field production and construction conditions. Numerous laboratory compaction methods have been developed in an attempt to accurately simulate field conditions. Some of these methods, such as the Hubbard-Field Method, have fallen out of favor, either because they fail to capture an accurate representation of field conditions or they have proven inefficient (Asphalt Institute [4]). Pavement engineers have long recognized that different compaction techniques produce specimens with different physical properties (Button et al. [6]), yet no consensus has evolved as to which method is best.

This chapter addresses this important aspect of mix design and provides data to compare various compaction methods. The review outlines and compares five of the major laboratory mixture and compaction methods still in use throughout the world: Marshall compaction method, Hveem mix design (kneading compaction method), French roller compaction method, German sector compaction method, and the Superpave gyratory compaction method. The report focuses on analyzing the ability of these compaction methods to simulate field compaction by comparing images of field compacted samples captured with a newly developed procedure with lab compacted samples of the same mixture taken from the field. The compaction was done at various laboratories around the world and the imaging was conducted by one operator who captured and analyzed the aggregate structure

and volumetric properties. The objective was to determine if the compaction procedure affected the aggregate structure within the compacted mixture and which compactor best resembled field compaction.

3.1.2 History of Hot Mix Asphalt Mix Design and Laboratory Compaction Methods

In the late 1860s, mixtures of sand, gravel, broken stone, ashes and coal tar binder were used in Brooklyn, New York and Washington, D.C. as the first recognized form of “rigid” pavement. At this time, natural rock-asphalt binder pavements were being developed for roadways in Paris, although the first true asphalt pavement was a sand-asphalt mixture laid in Newark, New Jersey in 1870. Not long after, similar mixtures were used in New York City, Philadelphia, and Washington D.C. Asphalt pavements continued to grow in popularity and roadways utilizing asphalt mixtures quickly spread (Goetz [14]).

Engineers soon realized that dense packing (road adequate compaction) provided resistance to “shoving” under traffic loading. Along with this, it was discovered that aggregates with jagged, sharp surfaces and of uniform gradation would better adhere to the asphalt and provide more uniform compaction. Accordingly, several laboratory methods were developed to satisfy the need for stable and durable field pavements; this meant engineers needed to duplicate field mixtures in the laboratory to accurately represent the mixtures as they exist in-situ to obtain a better understanding of the materials (Gartner [13]). Although existing laboratory methods vary in specimen preparation, compaction type (impact, kneading, gyratory, etc.), and procedure, the targeted final result is the same for each; the laboratory prepared specimen must be a useful representation of field conditions. The role of compaction in asphaltic mixtures was early recognized as crucial, as many mixture properties depend heavily on both degree/level and method of compaction (Asphalt Institute [4]).

3.1.3 Engineering Considerations in Asphalt Concrete Compaction

Perhaps the most important consideration in laboratory compaction is the approximation of field loading conditions, i.e. traffic weight and volume. The percentage of large axle loads (as well as the total number of loads) has increased dramatically in the last quarter century, requiring pavements capable of carrying this increased loading (Gartner [13]) and which requires better consideration of traffic conditions in the design process. Understanding of particular pavement failure mechanisms has led to the development of design specifications which directly

address pavement distresses such as rutting (Lee and Mahboub [20]), but it was recognized early on that the optimization of one quality (or resistance to a particular stress) may often come at the expense of another (Asphalt Institute [4]). Thus the mix design should be an optimization process that balances the resistance to distresses rather than focusing on one distress type. Each laboratory compaction method reviewed in this report attempts to isolate specific engineering properties in order to aid in predicting the response of a particular mixture to specific field conditions. For this reason, the underlying engineering principles will also be reviewed for each method.

3.2 Mixture Design Methods

3.2.1 Mixture Design and Compaction Methods

Hot mix asphalt densification in the lab and the field is influenced by factors including the method of compaction, physical properties of the components used in the mixture, sample geometry or lift thickness, construction conditions, and environmental conditions. This section of the report compares the methods of laboratory compaction and highlights the differences in some of these factors.

3.2.2 Marshall Design and Impact Hammer Compaction

The Marshall method of mix design and compaction was conceptually developed in 1939 by Bruce G. Marshall while he was working for the Mississippi Highway Department. The U.S. Army Corps of Engineers (USACE) revised Marshall's method during World War II for airfield pavement design. During that time, military aircraft wheel loads and tire pressures were increasing dramatically and the need for a simple apparatus suitable for use with the California Bearing Ratio equipment to design and control asphalt paving mixtures was realized. Throughout the 1950s, the USACE revised the Marshall Method by adding modifications such as a deformation measurement device and traffic loading variables. The resulting laboratory mix design and compaction procedure was adopted by the American Society for Testing and Materials (ASTM) and is still in use in many countries throughout the world. The Marshall method of mix design is empirical, with criteria based on the correlation of laboratory and field results (White [35]).

Preparation and compaction for the Marshall method of mix design follows ASTM D 6926-10: Standard Practice for Preparation of Bituminous Specimens Using Marshall Apparatus (ASTM [1]). Until 1989, ASTM D 1559 governed the stability and flow testing of Marshall specimens after compaction. This standard was then withdrawn with no replacement. Currently, ASTM D 6927-06: Standard



Fig. 3.1 Marshall impact hammers (Photo courtesy of PRI Asphalt Technologies, Inc.)

Test Method for Marshall Stability and Flow of Bituminous Mixtures is used to perform the test and calculations (ASTM 2010).

The compaction apparatus shown in Fig. 3.1 is comprised of three major components: the compaction pedestal and mold holder, the compaction mold, and the compaction hammer. The compaction pedestal is made of wood.

Compaction must take place within a range of temperatures that produce a binder viscosity of 0.28 ± 0.03 Pas. (ASTM 2010). The design traffic category of the mixture dictates to deliver 35, 50 or 75 impact blows on either side of the specimen.

After compaction, the specimens are subjected to density measurements, stability testing, and flow testing. Density is measured as both bulk specific gravity and maximum theoretical specific gravity. Specimens are then heated and circumferentially loaded until failure.

The major advantage of the Marshall compaction method is the focus on air voids analysis in the compacted specimens. Air voids analyses include voids in mineral aggregate (VMA), voids in total mixture (VTM), and voids filled with asphalt (VFA). Thorough analysis of air voids in the mix design is important to control two extreme distresses; asphalt flushing (bleeding) and premature aging of the asphalt pavements (Kandhal and Koehler [19]). The Marshall method recognizes the need for sufficient asphalt to ensure durability, sufficient stability to satisfy structural requirements under loading, sufficient air void content

(an upper limit to prevent environmental damage, such as moisture infiltration and excessive oxidation, and a lower limit to allow for additional densification from traffic), and workability to facilitate effective compaction (Lee and Mahboub [20]).

Air void content is a direct result of aggregate gradation, asphalt content, compaction effort, and compaction type. To achieve the optimum void content, the Marshall method utilizes impact loading in the form of a falling weight. The impact forces the aggregates to realign and form a more densely compacted specimen, thus decreasing air voids. Impact loading, however, may not create an aggregate structure that accurately simulates field compaction conditions (Button et al. [6]). Resistance to flow under traffic loading is considered using the stability and flow tests. Deformation under loading is recorded, and limited, in order to satisfy performance requirements for a given traffic load. However, the Marshall method fails to recognize the effect of shear strength on compacted specimens. Furthermore, specimens are loaded perpendicularly to the compaction direction, which differs from loading in the field (Lee and Mahboub [20]). Finally, it does not indicate potential tender mixtures during construction, rutting tendency under stationary loads and mixture fatigue characteristics and structural strength (Kandhal and Koehler [19]).

The Marshall mix design and compaction method was the primary method utilized by state agencies for nearly 50 years prior to the 1990s. About that time, researchers on behalf of the Federal Highway Administration began developing Superpave mix design to address problems with the Marshall design and other design methods. Today, Superpave mix design is the most widely accepted method of design in the US, although the Marshall method has been used in some capacity in at least 38 states (Davis and Dion [9]). It is also anticipated that even with the spreading acceptance of Superpave to replace older methods, the Marshall method will continue to be used for airfield pavement design.

3.2.3 California Kneading Compactor (Hveem Design Method)

During the late 1920s, the State of California began to pave rural roads with a mixture that was considered a compromise between high performance hot mix asphalt and asphalt oil sprayed on unbound particles. Materials and research engineer Francis N. Hveem developed a method to determine the amount of oil needed based on the total surface area of the aggregates, which was determined from the aggregate gradation. Hveem developed the Hveem stabilometer to measure this difference in stability with greater accuracy. Researchers soon recognized a difference in physical properties (stability) between specimens compacted in the field and those compacted in the laboratory. This led to the development of the modern day California kneading compactor, which was intended to more closely simulate compaction in the field (Vallerga and Lovering [34]).

Preparation and compaction using the California kneading compactor follows ASTM D 1561: Preparation of Bituminous Mixture Test Specimens by Means of

Fig. 3.2 California kneading compactor (Photo courtesy of Gilson Company, Inc.)



California kneading compactor (ASTM [2]). The compaction apparatus seen in Fig. 3.2 consists of a mechanical kneading compactor, mold holders, an insulated feeder trough, a trough paddle, and a round-nosed steel rod 9.5 mm in diameter and 406 mm long. The California kneading compactor utilizes a hydraulically operated tamper foot that applies pressure on a cylindrical sample. To simulate roller compaction, the tamper foot has a compaction face shaped like the sector of a circle. After each compression, the tamper foot lifts and the base of the compactor rotates the sample 1/6 of a revolution, or 60° (Asphalt Institute [4]).

The mold assembly is placed in position on the compactor and approximately 20 tamping blows at 1.7 MPa pressure are applied to achieve a semi-compacted specimen. This is done to ensure the full load will not excessively disturb the sample. The number of semi-compaction blows varies from 10 to 50 and is dictated by the type of material. Semi-compacted specimens are then adjusted on the compactor to allow for free up-and-down movement of the mold and about 3 mm of side-to-side movement. The compaction pressure is then increased to 3.45 MPa and 150 additional blows are applied.

In addition to a more refined compaction effort, the California kneading compactor and Hveem mix design method take into account absorption of asphalt by the aggregates, a previously overlooked phenomenon. Also, the strength parameters obtained from stability measurements provide a direct indication of the internal friction component of shear strength, an improvement over the Marshall method (FHWA [11]).

3.2.4 *French Roller Compactor*

The French roller compactor was developed by the Laboratoire Central des Ponts et Chaussées (LCPC). The LCPC (now IFSTTAR) is a French organization for the applied research and development of infrastructure and laboratory methods. Internationally involved with laboratory asphalt compaction since the 1950s, it has been instrumental in developing not only rolling wheel compaction, but also gyratory compaction, volumetric design, and failure mechanisms (Huber [16]).

Investigations into rolling wheel compaction began in the late 1970s with studies of densification characteristics related to compaction characteristics. The LCPC soon developed a unique method of mix design that standardized the compaction effort on the road, in contrast to the Marshall method that standardized compaction in the laboratory. The rolling wheel compactor was introduced to produce asphalt slabs that could be easily subdivided for subsequent testing, or tested in entirety (Huber [16]; EN 12697-33 [10]).

Sample preparation and compaction using the French roller compactor follows European Standard EN 12697-33: Test Methods for Hot Mix Asphalt: Specimen Prepared by Roller Compactor and French standard NF P 98-250.2: Preparation of bituminous mixtures.

Preparation of a sample follows a numerical equation to determine total mass of the bituminous mixture as a function of maximum density of the mixture, dimensions of the mold, and the voids content according to EN 12697-35.

The compaction apparatus, shown in Fig. 3.3, utilizes either one or two loaded, threadless (smooth) pneumatic rubber tires 400 mm in diameter and 80 mm in width moving back and forth to create asphalt slabs for laboratory testing. The single wheel compactor produces slabs measuring 500 mm $\times$ 180 mm for use in rutting tests, while the double wheel compactor produces slabs measuring 600 mm $\times$ 400 mm for modulus and fatigue testing. In both single tire and double tire compactors, the inflation pressure of the tires is held between 0.1 and 0.6 (± 0.03) MPa while applying a load of 1–10 kN on the specimen. The tire translation velocity varies based on testing conditions, but typically ranges from 200 to 500 mm/s, with one wheel pass defined as one-way movement of the tire. Compaction temperature is determined based on binder characteristics and grade. Generally, the compaction temperature is the temperature at which the binder viscosity is 200 mPa.s (EN 12697-33 [10]; Jönsson et al. [18]).

The compaction procedure depends on the specimen size and whether limiting compaction energy or limiting final bulk density is specified. If the compaction energy is specified, compaction procedures are broken down into one of two energy levels; light compaction for large and small slabs, and heavy compaction for large and small slabs. For each energy level, compaction of the specimen is carried out over three zones (Fig. 3.4) covering the width of the slab following a detailed sweep plan (detailed in European standards). To stabilize the specimen, each compaction zone is subjected to a pre-compaction of two wheel passes with low tire pressure (0.1 MPa) and low wheel load (1 kN). In the pre-compaction phase, the apparatus is



Fig. 3.3 LCPC French roller apparatus

set to blocked-axis mode where vertical motion of the tire is prohibited, thereby preventing the tire to compact the mixture below the level of the surrounding mold frame (EN 12697-33 [10]; Jönsson et al. [18]).

Effective compaction is carried out in a free-axis mode that allows the tire to move vertically, horizontally and transversely for transverse re-positioning of the tire in up-lifted position. Sweep plans follow an organized procedure; the compaction tire always starts in the front compaction zone, moves to the rear compaction

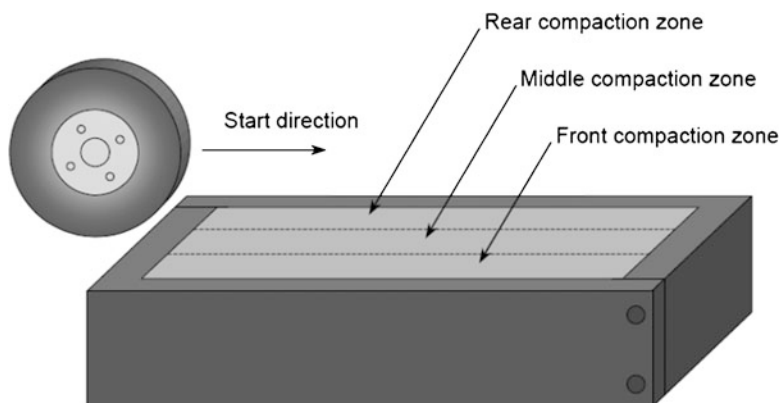


Fig. 3.4 Roller compaction zones

zone, and finishes in the middle zone. Before each compaction step, the compaction mold is lowered (or specimen is raised) so the top of the specimen is just above the mold frame. After the effective compaction has concluded, the specimen is subjected to a leveling, blocked-axis procedure consisting of two wheel passes over each zone using 0.6 MPa tire pressure and a wheel load of 5 kN (EN 12697-33 [10]; Jönsson et al. [18]). Start to finish, the compaction procedure takes 20–25 min to complete. The slabs are then left to cool at room temperature before removing the mold (done with light tapping from a hammer) (FHWA [12]).

Compaction of specimens to a specific voids content, or bulk density, requires modification of the compaction procedure. The user must choose a compaction procedure which yields specimens just above the desired void content and then begin to increase the number of passes until the specified density is reached. The sweep plane must remain even to ensure even compaction and a smooth surface. At the end of compaction, the user must report the testing/compaction procedure (EN 12697-33 [10]; Jönsson et al. [18]).

The purpose of rolling wheel compaction is to create samples that are representative of samples compacted by pneumatic roller compactors in the field. The French roller compacts a specimen in three distinct wheel paths, creating a kneading action between the aggregates of the sample similar to the kneading action induced in the field by pneumatic rollers. The kneading action allows coarse aggregates to realign and orient themselves in a particular fashion. However, unlike the California kneading compactor and the Superpave gyratory compactor, the French roller does not incorporate a static leveling load; rather it uses the pre-compaction phase of the wheel passes at reduced load. Static leveling may increase particle contact by bringing aggregates together (Muniandy et al. [27]). An advantage of the French Roller is that the specimen can be compacted in layers of realistic thickness, producing a sample that is dimensionally representative of that in the field. The mold can also accommodate larger aggregate sizes than other laboratory methods, again possibly allowing a mix design more closely related to the field

(Jönsson et al. [18]; Consuegra et al. [8]). Unfortunately, because of specimen size and geometry limitations, compaction at the start and end of the wheel path is not homogeneous. Hence, heterogeneity among front, middle and rear compacted zones may occur (Jönsson et al. [18]). This has to be taken into account when cutting samples from roller compacted specimens.

Although appealing because of its likeness to field compaction processes, laboratory application of the French roller and other similar rolling wheel compactors is very low for a number of reasons. Most laboratory rolling wheel compactors are comparatively large, not portable and relatively expensive. Difficulties in controlling air voids in compacted specimens have also been widely recognized. Moreover, compaction procedures are difficult to follow and very time consuming. Mixtures for compaction must also be prepared in comparatively large quantities, adding to the cost of laboratory investigations (Muniandy et al. [27]). In parts of Europe, rolling wheel compactors remain popular due to the improved representation of field compaction procedures and growing experience with the lengthy compaction procedures.

3.2.5 German Sector Compactor

The German sector compactor was conceptually developed in response to a research project funded by the Ministry of Transport at the Institute for Road Building in the Technical University of Braunschweig in the early 1990s. The aim of the project was to support the notion that mechanical characteristics of asphalt mixtures can be heavily influenced by the type of compaction in the laboratory, an idea first conceived at the end of the 1970s at TU Braunschweig (Renken [30]).

The result of the 1990s research project was a laboratory compactor that could produce asphalt slabs with tolerable mechanical properties for practical application in the field. The compactor utilizes a steel roller sector to compact samples within a heat chamber, using path-controlled pre-compaction and power-controlled main compaction features (Renken [30]). This compactor has since been referred to as the German (steel) sector compactor.

Sample preparation and compaction using the German sector compactor follows European Standard EN 12697-33: Test Methods for Hot Mix Asphalt: Specimen Prepared by Roller Compactor and similar applicable German standards.

Preparation of the mixture follows EN 12697-35, while a numerical equation is used to determine the total mass of the bituminous mixture as a function of maximum density of the mixture, dimensions of the mold, and the voids content. The compaction device (roller sector, mold and bottom steel plate) is pre-heated to a predetermined temperature and a lubricating solution or non-stick film is placed on the roller prior to compaction (EN 12697-33 [10]).

The compaction apparatus, seen in Fig. 3.5, utilizes a steel roller arc that is the sector of a circle with a radius of up to 550 mm (usually 500 or 550 mm) to apply a



Fig. 3.5 *Left: German sector compactor before use. Right: Sector compactor during use*

calculated downward force, or rolling force, on an asphalt slab to achieve a specified sample density. The rolling force can range from 0 to 30 kN. The magnitude of the downward force is calculated to achieve the specified volume or voids content of the final compacted sample after 10–50 roller passes. One roller pass is defined as the one-way movement of the steel roller. A frequency controlled motor shifts the mold left to right during the compaction process and compaction is done in a smooth continuous motion without interruption. Compaction temperature is again binder dependent, with a typical compaction temperature of $135 \pm 5^\circ\text{C}$ (EN 12697-33 [10]).

Compaction of samples using a roller sector can be accomplished by either compaction by a specified energy, compaction with a controlled compaction energy, or compaction to obtain a specified voids content and degree of compaction. Each compaction method follows a detailed pre-compaction and main compaction procedure where deflection and force are controlled, respectively.

The German sector compactor was developed to provide samples with mechanical characteristics comparable to those in the field. The compactor utilizes a steel arc to produce a kneading action and downward force to the specimen, intuitively appealing, given common field compaction methods. It was designed to simulate field compaction in both pre-compaction and main compaction phases. The pre-compaction is deflection-controlled and simulates the pre-compaction effect of the pressure bar on the paver screed and the tamper. The main compaction effort is force-controlled and simulates effective compaction by roller compactors in the field (Renken [30]). The sample does not experience any impact loading during compaction and, much like the French roller compactor, the German sector does not induce a static leveling load at the onset of the compaction routine. Static loads during leveling (pre-compaction) can move aggregates closer together, orienting them in a way that differs from field samples (Muniandy et al. [27]).

Laboratory use of the German sector compactor is relatively limited. The German sector compactor is a large, costly machine that is not easily transported. Also limiting the use and understanding of the German sector roller is the lack of available information regarding the roller and compaction procedure in English.

3.2.6 Superpave Gyratory Compactor

In the late 1980s, in an effort to combat the deteriorating highway infrastructure within the United States, congress authorized the Strategic Highway Research Program (SHRP). A major concern within SHRP was developing a laboratory compaction method that could compact specimens in the laboratory to densities measured in field pavements under traffic loading conditions. SHRP identified two major goals in the development of a compaction device: capability of measuring compactability in order to identify potentially tender mixture and problematic behavior, and adequate portability of the device for quality control purposes.

Building on the technologies of the three prevalent gyratory compactors in use at the time, the Texas gyratory compactor, French gyratory compactor, and the Army Corps of Engineers gyratory compactor, SHRP developed the Superpave gyratory compactor. Development of gyratory compaction as a principle was an iterative process utilizing properties of each gyratory compaction method. Alongside the Superpave gyratory compactor, Australia also developed a version of gyratory compaction worth noting (Bayomy et al. [5]; Harman et al. [15]).

Sample preparation and compaction using the Superpave gyratory compactor follows ASTM D 6925: Standard Test Method for Preparation and Determination of the Relative Density of Hot Mix Asphalt by Means of the Superpave Gyratory Compactor. The aggregate mixture and asphalt binder are heated to a mixing temperature range that will produce an un-aged binder kinematic viscosity of $170 \pm 20 \text{ mm}^2/\text{s}$. After conditioning the aggregate mixture and asphalt binder at the required mixing temperature, aggregates and asphalt binder are mixed to create a uniform distribution of asphalt binder within the mixture. After mixing, the loose aggregate-asphalt mixture is conditioned at the compaction temperature for 2 h, stirring at 1 h to maintain uniformity. The compaction temperature is defined as the temperature range that produces an un-aged binder kinematic viscosity of $280 \pm 30 \text{ mm}^2/\text{s}$. The compaction mold is also conditioned at the compaction temperature for no less than 45 min prior to the compaction process (ASTM [3]).

Following the conditioning period, the mold and mixture are removed from the oven and the mixture is transferred to the compaction mold. The mold is then loaded into the gyratory apparatus, shown in Fig. 3.6. Once loaded into the compactor, the apparatus will apply the required vertical pressure and angle of gyration and begin compacting. In general, the compactor operates with a constant vertical pressure of 600 kPa, with an angle of gyration of 1.25° and a gyration rate of 30 rpm. The top and bottom platens of the mold remain parallel throughout the compaction process; hence the angle of gyration revolves around the sample.



Fig. 3.6 Superpave gyratory compactor

The height of the specimen is automatically recorded with each gyration, allowing density calculations to be carried out for each gyration knowing the mass of the sample. Specimens are either compacted to a specified number of gyrations or compacted to a specified height for determination of volumetric or physical properties, respectively (ASTM [3]).

The Superpave gyratory compactor utilizes several important design principles to achieve these goals. It utilizes an angle of gyration of 1.25° to compact laboratory specimens. The use of a constant vertical pressure and constant angle of gyration allows the formulation of compaction curves. Tilting of the mold during the compaction process creates both normal and shear forces in the mixture. A vertical pressure of approximately 600 kPa is applied to the sample during compaction. The vertical pressure is an estimated equivalent to field loading conditions originally determined for use in the LCPC (French) gyratory as an approximation to field compaction using static rollers.

The number of gyrations used in compaction with the Superpave gyratory compactor is determined primarily by the design traffic level. Equivalent single axle loads (ESALs) are used to determine three compaction levels/criteria in the compaction process. The three levels are defined as N-initial (N_{ini}), N-design (N_{des}), and N-maximum (N_{max}) (Huber [16]).

In volumetric mix design, the asphalt content is selected to produce a specimen with 4% air voids at N_{des} . Four percent air voids represents the density of a mixture in the field during the service life of the pavement. N_{ini} is also selected based on traffic level and climate and primarily serves to identify tender mixtures (too easily compacted/densified). At N_{ini} , the specimen should have at least 11% air voids,

representative of air void content just after pavement construction. $N_{\max}$ is selected to represent the air voids content in the mixture at the terminal life of the design pavement. The air voids in the mixture at $N_{\max}$ are to be no less than 2%. Generally, multiple mixtures are designed to attempt to capture the optimum asphalt content – the asphalt content that satisfies the density requirements at each design gyrations number degrees (Huber [16]).

3.3 Hot Mix Asphalt Image Acquisition, Processing, and Analysis Procedures

This section includes a review and summary of the use of imaging technology as it has evolved through application to the hot mix asphalt field.

3.3.1 *Need for Imaging of HMA*

In order to quantify the internal structure characteristics of asphalt mixtures such as aggregate contact, air void, or aggregate segregation; an internal image should be obtained, processed and analyzed. Methods for acquiring an internal image of an asphalt mixture sample fall into two categories: destructive and non-destructive methods. Destructive methods require cutting the sample either horizontally or vertically to reveal the internal surface of the mixture and a picture of the surface is then obtained. Non-destructive methods are based on obtaining internal structure images by means of x-ray computed tomography (CT). Once the internal structure image is obtained, image processing is required to enhance the quality of the image and allow a clear definition of the image components (air voids, mastic, and coarse aggregates).

Several techniques have been used to process images of asphalt mixture internal structure over the past two decades. Image processing techniques can be manual, automated, or semi-automated.

- (a) Manual image processing techniques are time consuming and usually require identifying the perimeter of every coarse aggregate particle or changing their color intensity to assist in differentiating between the phases or components of the mixture.
- (b) Automated processing techniques involve using pre-written algorithms to apply image processing filters and operators. Although these methods are automated, they still require some pre-processing input, such as the volumetrics of the mixture.
- (c) Semi-automated processing methods are based on algorithms that apply image processing filters and operators to the image. The user is prompted or expected to change the numerical value of such filters and operators when different

images are analyzed. The user will check the value of certain measured parameters against the known real values for, for instance, the air voids content.

The last step is to analyze the processed image to obtain meaningful parameters that describe the internal structure of the mixture. Parameters such as aggregate size distribution, orientation, segregation, and contact points, and air voids distribution are the main parameters that can be obtained from such analyses.

Several studies have focused on the different methods used to acquire, process, and analyze images of the asphalt mixture structure. The studies included here were selected to cover the most commonly used imaging techniques in asphalt mixtures since 1995 and by no means cover every study conducted since the mid 1990s.

3.3.2 *Early Innovation*

3.3.2.1 **Defining Internal Parameters**

Yue et al. [36] published one of the first comprehensive studies on defining internal aggregate parameters derived from imaging. Major and minor axes of aggregate particles are essential for quantifying both aggregate shape and orientation. The major axis is the longest distance between two pixels of the boundary contour and that distance is defined as the major axis length. The minor axis is the longest line that can be drawn perpendicularly to the major axis and the distance between the two end points of the minor axis is defined as the minor axis length. For each aggregate particle, the following parameters are calculated: perimeter, area, centroid, Feret diameter, major axis orientation, shape factor, and compactness (Yue et al. [36]). Aggregate particle perimeter is the summation of all pixels forming the aggregate boundary. The aggregate area is the summation of all pixels within the aggregate particle and the centroid is the coordinates of the center of mass of the aggregate. The Feret diameter is $(4 \times \text{area}/\pi)^{0.5}$ and is defined as the diameter of a fictitious circular aggregate that has the same area as the aggregate. The major axis orientation is the angle between the major axis and the x-axis.

Shape factor and compactness are measures of the aggregate particle shape. The shape factor is a measure of how circular an aggregate cross section is. It is equal to $(4\pi \times \text{area})/(\text{perimeter}^2)$ and ranges from 0 to 1, where 1 represents a perfect circle. Compactness is a measure of aggregate cross section circularity and is calculated as $\text{perimeter}^2/\text{area}$. It ranges from 12.6 to infinity (a perfect circle represented as 12.6).

In addition to these parameters, statistical and gradation parameters have been introduced to study internal structure. Statistical parameters include:

1. *Area percentage*: percent of the area occupied by all aggregates over the total area of the asphalt concrete (AC) section,
2. *Centroid of aggregates*: the centroid of all aggregates in the AC cross section,

3. *Eccentricity*: percentage of the distance of the centroid of aggregates to the geometric center of the AC section over the radius (or width) of the AC cross section,
4. *Percentage of moment of inertia*: percentage of the summation of the moment of inertia of the aggregates over the total moment of inertia of the AC cross section.

Three gradation analysis methods are using these internal parameters:

1. *Area gradation by minor axis length, Feret diameter, or major axis length*: the sum of the area of aggregates that has a minor axis length, Feret diameter, or major axis length in the range of 2 mm to a dimension d ($d \geq 2$ mm), divided by the total area of all aggregates. This gradation is expressed as a percentage function of d .
2. *Number (or area) gradation by shape factor*: the sum of the number, or area, of aggregates with shape factor values between 0 and s divided by the total number, or area, of all aggregates. The value s ranges from 0 to 1 and the gradation is expressed as a percentage function of s .
3. *Number (or area compactness) gradation by orientation*: the sum of the number, area, or compactness for all aggregate particles with major axis orientation angle between -90 and (θ) over the total aggregate number, area, or compactness of all aggregates in the section. Theta (θ) is in the range -90 to $+90$ and the gradation is expressed as a percentage function of θ .

3.3.2.2 Ease of Digitization

Three years after the defining of many internal parameters, another study (Masad et al. [25]) focused on reducing the labor intensive procedures established by Yue et al. [36] to make imaging of HMA more appealing. In this study, asphalt samples compacted using the gyratory compactor in the laboratory and from field cores were cut vertically using a diamond saw at a slow cutting rate to ensure smoothness of the surface. The samples were cut into three equally spaced sections (37.5 mm apart) as shown in Fig. 3.7. A digital camera was used to capture images at each of the three sections. Thresholding was used to separate the aggregate particles from the other phases of the asphalt mixture (binder and air voids). However, manual editing by painting the dark aggregates white was necessary to enhance the contrast and ease the thresholding process. An image tool program was used to edit the images in order to separate conjoined aggregate.

The digital images were analyzed to quantify aggregate orientation and segregation in the asphalt mixture. Aggregate particle orientation is the angle between the particle major axis and the horizontal axis, the same major axis orientation established by Yue et al. [36]. Particles in contact with the edges were discounted from the analysis to avoid a boundary effect on the results. The major axis of aggregate particles was defined as the axis connecting the two pixels on the particle boundary with the largest separation distance, again consistent with Yue et al. [36]. After determining the orientation angle for all the aggregate particles in the image,

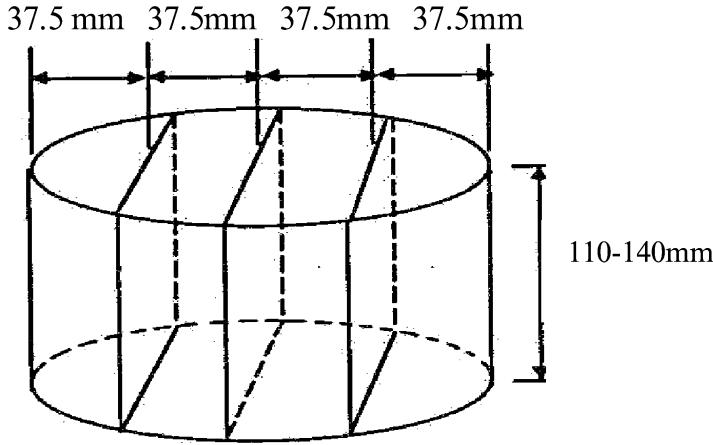


Fig. 3.7 Asphalt sample vertical sections location (Masad et al. [25])

two parameters were calculated to measure the directional distribution of the particles; the average orientation angle (θ) and the vector magnitude (Δ), defined to be between 0 and 100, as follows:

$$\Delta = \frac{100}{N} \left[\left(\sum_{k=1}^N \cos 2\theta_k \right)^2 + \left(\sum_{i=1}^N \sin 2\theta_k \right)^2 \right]^{\frac{1}{2}} \quad (3.1)$$

$$\theta = \frac{\sum_{k=1}^N |\theta_k|}{N} \quad (3.2)$$

where θ_k is the orientation angle for an aggregate particle, the range of θ_k is -90° to $+90^\circ$ (Fig. 3.8), and N is the total number of aggregate particles in the image. For a complete random distribution, the value of Δ approaches 0, while a value of 100 indicates that all the particles have the same direction. In addition, spherical harmonic series described the directional distribution of aggregate orientation as follows:

$$n(\theta_k) = n_a (1 + A_2 \cos^2 \theta_k + 2B_2 \sin \theta_k \cos \theta_k - A_2 \sin^2 \theta_k) \pi r^2 \quad (3.3)$$

$$n_a = \frac{N}{N_d} \quad (3.4)$$

where $n(\theta_k)$ is the number of aggregate particles oriented at an angle of θ_k from the horizontal, and n_a is the number of aggregates randomly oriented within an image. The directional function parameters A_2 , and B_2 were computed as:

$$A_2 = \frac{2 \sum_{k=1}^N \cos (2\theta_k)}{N} \quad (3.5)$$

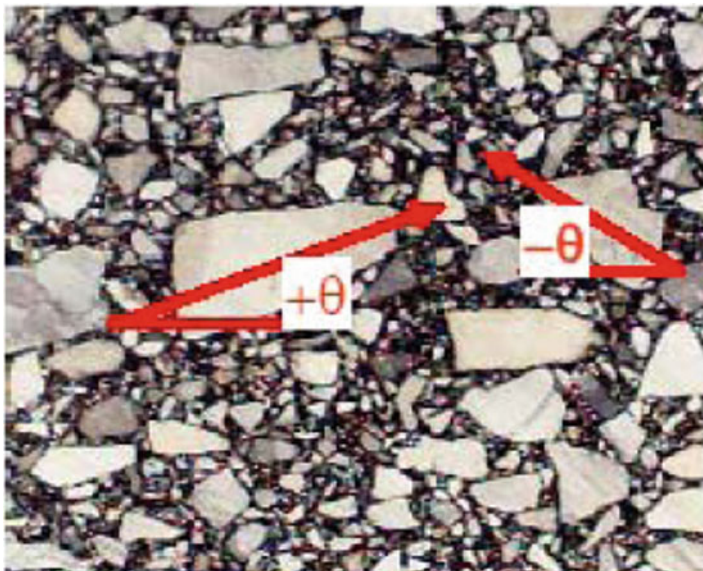


Fig. 3.8 Angle of inclination (θ_k) (Masad et al. [25])

$$B_2 = \frac{2 \sum_{k=1}^N \sin(2\theta_k)}{N} \quad (3.6)$$

N_d in Eq. (3.4) is the number of angles (θ_k) considered in the spherical harmonics series. In this specific study, an N_d value of 36 was used.

Segregation was a newly defined parameter for internal structure images of HMA. Segregation was defined as the separation between the fine and coarse portions of the asphalt mixture. Two procedures were used to determine segregation. In the first procedure, asphalt mixture images were divided into four quadrants. The segregation parameter was calculated as the coefficient of variation of the average aggregate diameter in the four quadrants. In the second procedure, a random quadrant sampling approach was used. One hundred random sampling quadrants were then distributed within the asphalt mixture image.

3.3.3 Application of Imaging in HMA Compaction Studies

Shortly after the work of 1998, Masad's team continued their contribution to digital imaging of HMA with two follow-up studies in 1999. In early 2009, the Texas Transportation Institute (TTI) published a report summarizing an extensive literature review related to a project entitled Application of Imaging Technology to Improve the Laboratory and Field Compaction of HMA [25]. This literature

review is relevant to the discussion of image acquisition, processing, and analysis procedures as covered in this report. The TTI report covered critical developments during the period of 1999 through 2001. The research review is further summarized here.

3.3.3.1 Advances in Non-destructive Imaging

Building on earlier work by Masad et al. [22–24] using 2D imaging, computed tomography (CT) scanning was introduced in new studies as a non-destructive imaging method to obtain the internal structure of asphalt mixtures. In this approach, a specimen is positioned between the x-ray source and the detector and the system is activated. A CT image is produced by passing x-rays through the specimen along several paths in different directions. The specimen is rotated about its center and the process of scanning one slice is complete after collecting the measurements for a full 360° rotation. The x-ray intensity is measured before entering and after passing through the specimen. The next step is to shift the specimen vertically to scan the next horizontal slice, repeating the entire procedure. The vertical shift represents the thickness of the scanned slice. CT images represent a spatial distribution of density. Image processing and analysis is then conducted to extract useful information from the image.

CT images consist of 256 gray intensity levels. Different levels correspond to different densities within the image (i.e., asphalt specimen). CT images can be analyzed to obtain the air voids distribution. Air voids density is lower than other components of asphalt mixtures, is a suitable threshold of gray intensity level and can be used to separate air voids from the other components (aggregates and mastic). When threshold is applied to the image, all pixels with gray intensity level values lower than the threshold are considered air voids, while the rest of the pixels (gray intensity level higher than the threshold) are considered aggregate and mastic. In other words, a 256 gray intensity levels image will become a binary image, where white pixels represent air voids and black pixels represent aggregate and mastic. Thus, the air voids percentage can be calculated for the image. The air voids analysis is based on finding the air void percentages in the different X-ray slices, and thus the air voids vertical distribution can be described.

A method for evaluating the number of contacts for 2D images of asphalt mixtures is also introduced. Erosion is the main operation by which contacts between aggregates can be evaluated. It is a morphological operation in which certain pixels are removed from the image based on the number of pixels of differing colors that surround that pixel. When two objects appear to be in contact within an image, black pixels are surrounded by white pixels. When erosion is applied, the black pixels at the contact points are removed. This leads to a reduction in the Particle size by eliminating the counting of two adjacent aggregates as one, and removing small particles after a number of erosion iterations.

3.3.3.2 Improved Resolution and Automation

CT was used for the investigation of the internal structure of HMA, which prompted further study developing advanced techniques to improve the resolution, which is limited by CT (Tashman et al. [32]). Tashman et al. developed a system that includes three main steps: image acquisition, image processing, and image analysis. The internal structure is captured both by x-ray images and by physically cutting asphalt samples horizontally and vertically. The air void distribution and aggregate orientation are calculated for the x-ray images in a manner similar to the previous studies (Masad et al. [23, 24]), however, a new parameter for aggregate orientation was introduced, the average angle (AA). The purpose of this new parameter is to find the average orientation angle, including the effect of the sign of the angle, rather than the absolute average used by Masad et al. [23, 24], although the absolute average was also noted in the study.

A new concept for measuring aggregate segregation was introduced in this study and the following definition was adapted for segregation: “lack of homogeneity in the asphalt mixture constituents (asphalt, fine aggregate and coarse aggregate) of the in-place pavement layer of such a magnitude that there is a reasonable expectation of accelerated pavement distresses”. In order to quantify aggregate segregation, the lateral separation between coarse and fine aggregates was measured by dividing the image into inner and outer regions. The segregation parameter (SL) was defined as the percent increase of the average aggregate particles diameter in the outer region compared to that in the inner region. The segregation index value is either negative, positive, or zero. A negative SL value indicates more coarse aggregate distributed in the inner region, while a positive value indicates that more coarse aggregate is distributed in the outer region. A zero value indicates that coarse aggregate are equally distributed within the two regions, i.e., no lateral segregation.

Aggregate contacts within the asphalt mixture were automatically quantified. The original image was converted into segments of lines, each line representing the regions of contact among the aggregate particles. The number of lines represents the number of contacts within the mixture. The contact normal was then calculated by rotating each line 90° and was then further analyzed by calculating AA and vector magnitude.

3.3.4 Advanced Analyses

3.3.4.1 Expansion of Segregation Parameters

A 2004 study maintained the less labor-intensive approach established earlier and physically cut HMA samples to capture images directly from the resulting cut surfaces (Hunter et al. [17]). Cylindrical laboratory specimens were cut using a

diamond-edge circular saw at two locations (first cut 20 mm from top of specimen and second cut 20 mm from bottom of specimen). The images were captured using a digital camera. Image Pro software was then used for the image analysis. White aggregate was selected to assure contrast between the aggregate and the asphalt binder in the images. After obtaining the images, thresholding was applied to distinguish between aggregate particles and the asphalt binder. The analysis did not include air voids. Based on volumetric calculations, the threshold used in this study could not recognize 25% of the total aggregate particles. This percentage was deemed acceptable because only aggregates of large sizes were thought to greatly affect the aggregate structure.

Once the aggregate particles were distinguished, the software was used to calculate the particle cross-sectional area, center of the particle cross-sectional area, particle vertical angle B (the angle between the particle major axis and the vertical axis), particle maximum length, and particle maximum width (each of which is comparable to the previously defined major axis orientation, major axis length, and minor axis length, respectively). Aggregate structure within the asphalt mixture was described by quantifying the aggregate orientation and the aggregate segregation. Aggregate orientation (comparable to the previously defined aggregate orientation angle) was estimated by calculating a weighted average of the circumferential aggregate orientation. The aggregate particles were split into 31 groups based on their area. The orientation was calculated for each range separately.

Aggregate segregation was estimated following two different approaches: radial segregation, which is relative to the cross section center, and regional segregation, which divides the core cross section into inner and outer regions. Radial segregation is based on splitting the cross section into sectors. Five degree sectors were used in this study and, thus, 72 sectors covered the entire cross section. Any 18 adjacent sectors will represent one quarter of the sample. The quarter with the maximum aggregate area was then selected and the aggregate area in the other three quarters calculated. The assumption is that, if the aggregate particles are evenly distributed in the mixture, then the four different quarters should have a relatively close aggregate area. Similarly to the orientation approach, segregation can be calculated for all aggregate sizes or for specific sizes. Regional segregation is based on the area of the aggregate in each region, where the ratio of the aggregate area (inner/outer) is equal to 1.0 for evenly distributed aggregates.

3.3.4.2 Air Void Distribution

A 2009 study (Thyagarajan et al. [33]) focused on comparing the air void distribution for asphalt samples compacted using different gyratory compactors. Air voids were found as explained by Masad et al. [23, 24], however, new parameters were introduced to quantify the air void distribution: vertical heterogeneity index (HI_{ver}) and lateral heterogeneity index (HI_{lat}). Heterogeneity indices reflect the level of heterogeneity in the air void distribution in a specific direction (vertical or lateral). This is determined by calculating the deviation of the air void distribution from

an average air void value (i.e., homogeneous). High values of heterogeneity index indicate high heterogeneity in the air void distribution, while an index of zero indicates a perfectly homogenous sample. The vertical heterogeneity index was calculated using Eq. 3.7. In this equation V_i is the average air void content of k successive x-ray images. The reasoning behind using an average of successive air void contents was that the horizontal planar scanning took place at 0.2 mm intervals and this caused sharp changes in the vertical air void distribution curve. Such sharp changes will influence the heterogeneity index. Other parameters in the equation are defined as follow: V_{Avg} is the air void content for the sample (lab measurement); T is equal to N/k , where N is the total number of x-ray images for the sample.

$$HI_{Ver} = \frac{1}{T} \sum_{i=1}^T abs\left(\frac{V_i - V_{avg}}{V_{avg}}\right) \quad (3.7)$$

The relationship between k and HI_{Ver} was studied and the heterogeneity index decreased with an increase in k ; the decrease was steep at low values of k , but then reached a steady state. The percent of k/N was used instead of k to eliminate the influence of the sample height. A k/N value of 1.5% was selected to effectively eliminate the influence of the sharp variations on the heterogeneity index. The value of k changes from one sample to another based on its height, keeping k/N constant.

In order to calculate the lateral heterogeneity index, the x-ray images were divided into four regions (equal in cross sectional area). The heterogeneity index was then calculated as follows:

$$HI_{Lat} = \frac{1}{M} \sum_{i=1}^M abs\left(\frac{V_i - V_{avg}}{V_{avg}}\right) \quad (3.8)$$

where M is the total number of the regions (four in this specific study); V_i is the air void content in the i th region; V_{Avg} is the air void content for the sample (lab measurement). It important to mention that V_i was measured along the entire height of the sample.

3.3.4.3 Full Automation

The reduced time/effort of obtaining several slices to image from a given sample became accepted procedure, so recent studies turned efforts toward easing the complexity of processing and analyzing the captured images (Zeleeuw and Papagiannakis [37, 38]). The 2009 study introduced an automated digital image processing technique for analyzing the internal structure of asphalt mixtures from CT images. MATLABTM was used to implement the different stages of this technique. The volumetrics of the asphalt mixture sample was required to process the CT images. The approach was based on finding two gray level thresholds to distinguish air voids, mastic, and aggregates. The approach consisted of three

separate stages: Stage I is image pre-processing for contrast enhancement and noise removal, Stage II is the main volumetrics-driven thresholding routine, and Stage III is the enhancement of aggregate particles separation through edge detection and image segmentation.

Stage I: Contrast enhancement and noise removal: Histogram equalization (or linearization) was used to enhance the image contrast. In this method, the gray level intensity for the image pixels was adjusted to produce a more uniform distribution throughout the image. The built-in MATLABTM function *histeq* was used to achieve this part. Median filtering was used to assist in removing the noise associated with CT images. The median filtering entailed replacing the gray level of each pixel by the median of the gray level of all pixels in its neighborhood. In this study a 3×3 window was used to define the local neighborhood.

Stage II: Volumetrics-based thresholding algorithm: The basic idea behind this algorithm was to identify two different thresholds, the first one to distinguish air voids from the rest of the asphalt mixture (aggregates and asphalt binder) and the second to distinguish the aggregate from the mastic (air voids, asphalt binder, and fine aggregates). The second threshold value was estimated following the same procedure. However, the threshold value targets separating the aggregate particles from the rest of the mixture and, thus, the percent of aggregate area was calculated, rather than the air voids.

Stage III: Edge detection and image segmentation: One of the techniques for enhancing the boundaries between different phases in a digital image is edge detection. This was achieved by using a gradient approach. The gradient of an image was defined as the value of the first derivative of its gray level and the image gray level was treated as a two variable function $f(i,j)$. The Canny operator method was used to implement this approach. In this method, the first derivative of gray intensity was calculated, but only specific derivatives that exceed a threshold value were retained. After calculating the gradients, a Gaussian filter was used to identify significance. Image segmentation by watershed image segmentation was used to further enhance the separation between particles that touch each other. Morphological dilation and erosion were applied to the images to perform the watershed technique. As in all stages of this method, MATLABTM was used to apply the watershed.

After completion of the three stages, the resulting representations of the asphalt mixture effectively identified the three phases within the mixture, air void, aggregate, and mastic. Such a representation could be used directly in numerical simulations for microstructure modeling purposes.

In a follow up study by same authors (Zezelew and Papagiannakis [37, 38]), discrete wavelet transform (DWT) was applied to images processed by this algorithm in order to measure segregation within asphalt mixtures. DWT was used to analyze the distribution of aggregates of different sizes in the images. A segregation index was defined to measure the coarse and fine aggregate segregation in both horizontal and vertical directions.

3.3.5 Development of an Imaging Standard

As part of the RILEM TG2, an effort was made to review available imaging methods and identify gaps in the evolution of imaging techniques for asphalt mixtures. It was clear that there are many good ideas but that the details of imaging, processing, and analysis lack standardization. Effort was expended to develop a standard procedure that captures the best ideas available for a practical and effective imaging technique. As a result, a new procedure for the imaging and analysis of 2D sections of asphalt mixtures was developed (Coenen et al. [7]).

An attempt was made to standardize the technique and the procedure was written up in a draft AASHTO format entitled “Determining Aggregate Structure in Asphalt Mixtures by Means of Planar Imaging”. The draft standard was written to summarize the image processing and analysis techniques incorporated in recently developed iPas software. This software was developed as collaboration between the University of Wisconsin at Madison and Michigan State University.

The iPas software allows analysis of 2D images accompanied by specific input data to calculate the number of aggregate contact points, aggregate orientation and aggregate segregation. The first step is to calculate the image resolution. The number of pixels along a known distance (usually the sample dimension) is used in the calculation. Image resolution is simply the distance (mm) divided by the number of pixels.

Volumetrics of the mixture is a required input of the software; they are used to calculate volume percentage of the aggregate by dividing the aggregate volume by the total volume of the mixture. The next step is to calculate the coarse aggregate fraction (CF) with respect to the total aggregate volume. In order to carry out this calculation, the software requires the user to identify the minimum size of the visible aggregate in the image. The software provides a tool for the user to draw a line along the smallest aggregate identified and, based on the image resolution, the minimum size is then calculated (D_{min}). Based on this minimum size, CF is calculated by dividing the volume of aggregate larger than D_{min} by the total volume of the aggregate. The aggregate volumes used in calculating CF are from the mixture volumetrics, not the image.

A series of image processing filters are then applied to the image (Fig. 3.9a) to separate the coarse aggregate particles from the rest of the mixture (binder, fine aggregates, and air voids). The first filter is the median filter, which is applied to remove random noise in the image (Fig. 3.9b). The next filtering process is applying the H_{max} filter to eliminate the variation in the aggregate pixel intensity (Fig. 3.9c). In order to separate the aggregate particles, the image is inverted and a watershed transformation is applied (Fig. 3.9d). A threshold is then applied to convert the image to binary scale (Fig. 3.9e). At this point, white pixels represent the aggregate part of the mixture. The final step in the processing procedure is to perform a binary labeling operation (Fig. 3.9f).

The image processing steps described above require the user to select the values for the median filter, H_{max} filter, and the threshold value. In order to verify that the

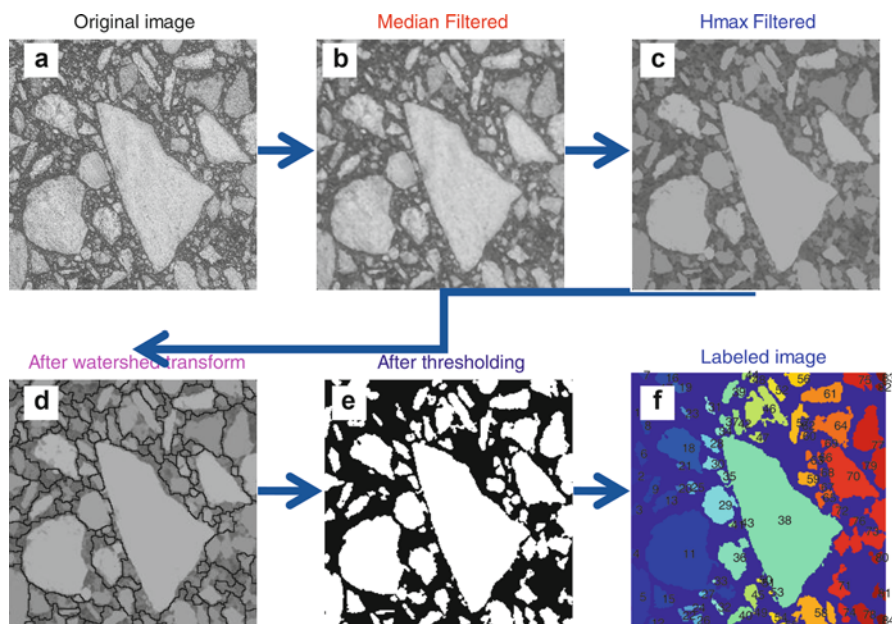
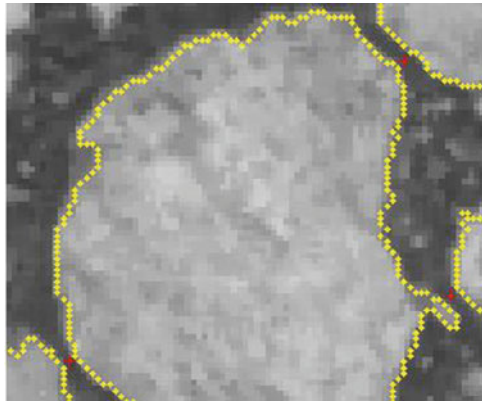


Fig. 3.9 Image processing procedure (iPas Software)

selected values are reasonable, the software calculates the area and equivalent diameter for all the labeled coarse aggregates. The gradation of the coarse aggregates in the image, excluding particles with diameters less than D_{min} , is then calculated, along with the percentage of coarse aggregate in the image. These two measures are then compared to their respective known values from the mixture properties. An iterative process is used until selected values result in image-based calculations matching known mixture properties. This iterative process replaces an otherwise automated process, now allowing users to determine if an image is accurately processed.

As mentioned above, the software calculates the number of aggregate contact points, aggregate orientation, and aggregate segregation. The contact points are calculated by measuring the minimum distance between the surface pixels of adjacent aggregate particles. If this distance is less than the pre-defined distance (surface distance threshold; SDT) then the two aggregates are considered in contact. Figure 3.10 shows an example of contact points calculated for adjacent aggregate. To eliminate any bias that may be inherent in a fully automated system, the iPas software allows the user to adjust the SDT to determine which aggregate proximity may be more relevant than direct contact, depending on the gradation or type of the asphalt mixture. In order to provide features consistent with previous developments, iPas also allows for the determination of aggregate orientation and segregation based on particle size.

Fig. 3.10 Illustration of aggregate contact (iPas Software)



3.3.6 Summary

Using imaging to capture aggregate structure and voids structure of asphalt mixtures has evolved significantly in the last 10–15 years. While the tools for imaging, such as 2D scanners and 3D CT scanners, have become readily available, the details of the best practice to capture an image, process it, and analyze it continued to vary and be debated. It is clear that for imaging to become a standard tool in asphalt mixture characterization, it has to be standardized similarly to the other characterization methods used in mechanical or volumetric testing. The work by the RILEM TG2 group has attempted to achieve this by developing standard software and a standard protocol for using the software. The protocol includes processing and analysis steps that utilize the best known filtering and verification steps for 2D and 3D images. The protocol includes calculating parameters well known in the literature and are considered effective in defining aggregate structure and aggregate interactions.

3.4 Effect of Compaction Methods on Internal Structure and Air Void Distribution of Asphalt Mixture

It is well recognized by pavement engineers that asphalt mixture internal structure, as identified by air void distribution, aggregate orientation, and aggregate contacts, has an important influence on its mechanical properties. Different laboratory and field compaction methods can produce specimens with different internal structures, and, hence, mechanical properties. The adequacy of any one laboratory compaction method in approximating field conditions requires a mechanical comparison. Several studies have been conducted to investigate and compare the effects of different laboratory compaction methods on mixture mechanical properties. There is, however, a limited number of studies that focus on the effect of

compaction methods on the internal structure of mixtures compacted under varying conditions. This section summarizes the findings of some of the important studies in this topic.

3.4.1 Marshall and Gyratory Compaction

When the Superpave gyratory compactor was introduced in the United States to replace the Marshall compactor, there was a significant interest in understanding how this change would affect the microstructure of mixtures. Yue et al. [36] studied laboratory and field mixtures produced by these two compaction methods. Laboratory specimens 100 mm in diameter and 50 mm in height were prepared using gyratory and Marshall compactors. Gyratory samples were compacted under 600 kPa pressure to a total of 250 gyrations at a 1° gyration angle. The Marshall samples were prepared using a standard Marshall compactor with 100 impact blows.

Field samples were retrieved from two experimental pavement sections that were compacted 4 years prior to this study. The cores were 95 mm in diameter and 50 mm height. One section was compacted using a conventional vibratory steel roller followed by a rubber-tire roller and the other section was compacted using an asphalt multi integrated roller (AMIR). AMIR was developed as a rubber composite roller that provides a softer compaction surface than the conventional stiff steel cylinder. This was expected to prevent compaction-induced surface cracking in the asphalt pavement.

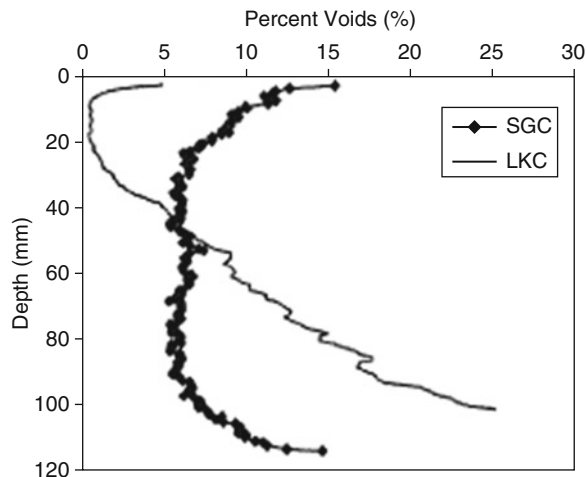
The microstructure of the asphalt mixtures was defined as the “internal structure of coarse aggregate distribution”. The area gradation of the coarse aggregates was based on rotary minor axis length, Feret diameter, and major length. The area percentage of aggregates; eccentricity of aggregates; and the moment of inertia with respect to the X and Y axes were the measures of the internal structure in this study.

The study concluded that asphalt mixture samples prepared by the gyratory compactor in the lab and the AMIR compactor in the field had more uniform distribution of coarse aggregate compared to the Marshall compactor in the lab and the conventional roller compactor in the field.

3.4.2 Gyratory and Kneading Compaction

The focus of a study by Masad et al. [23] was to capture the HMA internal structure for asphalt mixtures prepared using the Superpave gyratory and the linear kneading compactor (LKC). The internal structure was described by calculating the aggregate contacts, aggregate orientation, and air void distribution. Digital images of specimen sections were analyzed for aggregate contact and orientation and CT

Fig. 3.11 Air void distribution in SGC and LKC specimens (Masad et al. [23])



images were used for quantifying the air void. The following results were concluded in this study:

- Both compaction method (SGC and LKC) samples have non-uniform air void distributions. Figure 3.11 illustrates the air void distribution for specimens from the two compaction methods. The air void percentage increased by depth for the LKC specimen, while the SGC had a higher air void percentage at the top and bottom than in the middle.
- The air void percentages calculated from the analyzed CT images correlated well with the air voids measured in the laboratory.
- The LKC specimen had a higher number of contacts and a more random aggregate orientation distribution than did the SGC. The SGC samples exhibited a more horizontal aggregate orientation.

In a follow up study (Masad et al. [24]), the internal structure of asphalt samples prepared using the Superpave gyratory compactor was compared to that of samples from field cores. The Superpave samples were prepared at six compaction levels: 8, 50, 100, 109, 150, and 174 gyrations. The internal structure parameters quantified in this study were air void distribution, aggregate orientation, and gradation.

Figure 3.12 represents the air void distribution results for the gyratory compacted samples at different levels of compaction. The air void distribution of the field cores is shown in Fig. 3.13. The gyratory sample, like the previous study, showed a higher air void percentage at the top and bottom of the specimen. Also, the middle part compaction level increased as the number of gyrations increased. The air void distribution in the field cores was different from those of gyratory samples.

Figure 3.14 summarizes the average angle of orientation, vector magnitude, and percent of air void for the gyratory compacted specimen and field cores. The results indicated that the percent air void decreased with an increase in the number of

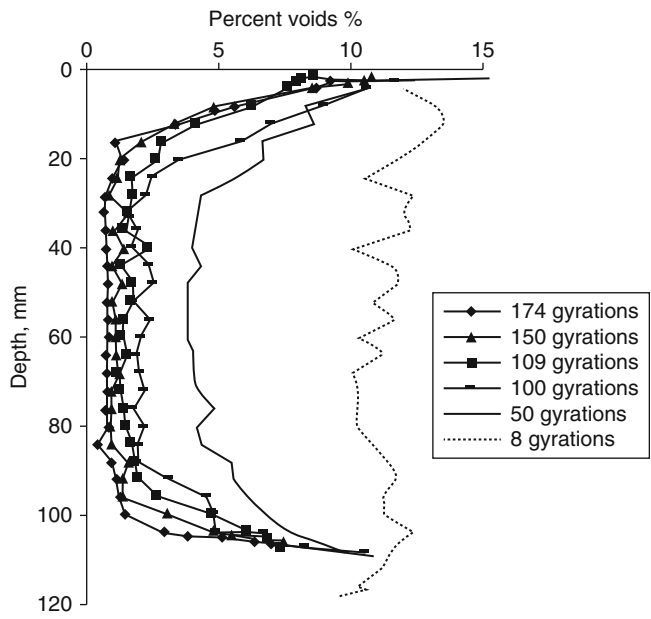


Fig. 3.12 Distribution of voids in gyratory specimens (Masad et al. [24])

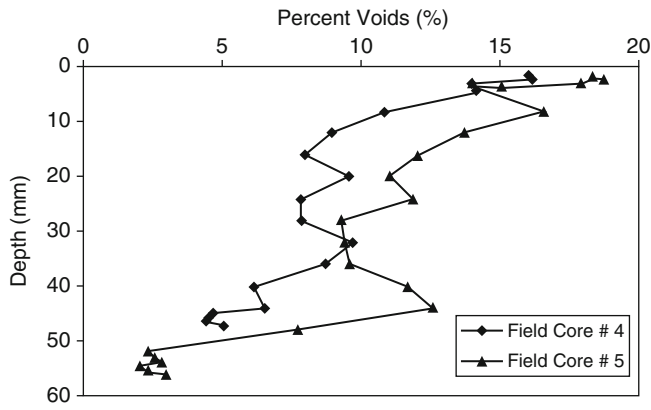
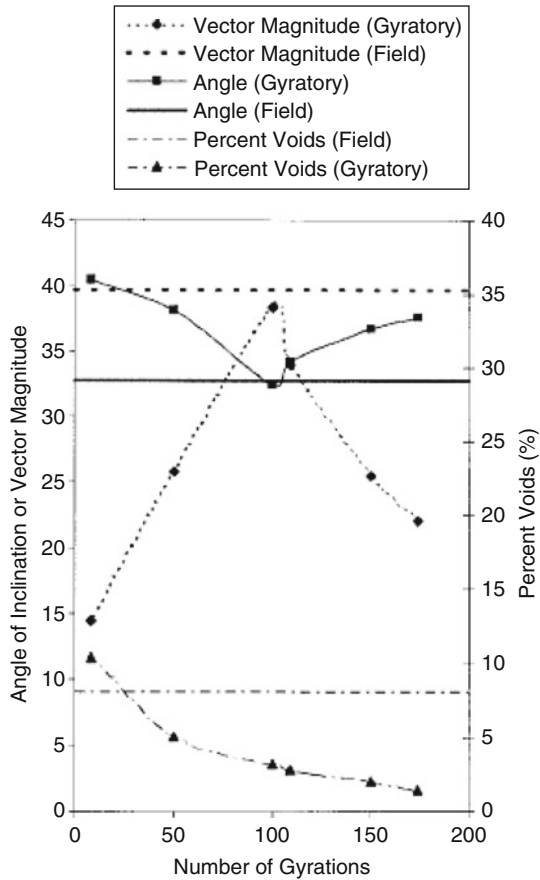


Fig. 3.13 Distribution of voids in field cores (Masad et al. [24])

gyrations up to a certain level of compaction, whereas the orientation angle increased after that compaction level. Conversely, the vector magnitude increased until it reached the same compaction level and decreased afterward. Also, the preferred aggregate orientation increased with increasing number of gyrations until it reached its maximum value. The level of gyrations at the maximum preferred aggregate orientation angle is called the optimum compaction level.

Fig. 3.14 Variation of vector magnitude, angle of inclination, and percent void with compaction (Masad et al. [24])



This was about 100 gyrations in this study. Further compaction after the optimum compaction level caused the aggregate to become more oriented and lose the preferred orientation angle.

3.4.3 Comparison of Gyratory and Field Compaction

Tashman et al. [32] studied the ability of the Superpave gyratory compactor to produce asphalt mixture specimens with internal structures that closely simulate field core internal structure. In this study, the laboratory compaction conditions were varied and, in the field, the effect of different compaction patterns on the internal structure was studied in detail. Table 3.1 summarizes the different compaction scenarios in the laboratory. The field compaction patterns are summarized in Table 3.2.

Table 3.1 Compaction scenarios in the laboratory (Tashman et al. [32])

Sample label	Angle (deg)	Height (mm)	Pressure (kPa)	Temperature of base plates and mold (°C)	No. of gyrations	Air void (%)
LHCC	1.25	135	600	149	31	7.71
LLCC	1.25	50	600	149	40	7.30
HHCC	2	135	600	149	13	7.15
LHHC	1.25	135	800	149	17	7.73
LLCH	1.25	50	600	175	47	6.45

Table 3.2 Field compaction patterns (Tashman et al. [32])

Test section	Breakdown (initial) compaction 149–116 °C	Intermediate rolling 116–88 °C	Finish rolling 88–1 °C	Average thickness of cores (mm)
1	Vibratory Steel Roller Ingersol Rand DD130 4 passes	Pneumatic Rubber-Tired Bomag BW20R 4 passes	Static Steel Wheel Roller CAT 534 C 4 passes	49
2	Vibratory Steel Roller Ingersol Rand DD130 4 passes	Vibratory Steel Wheel Roller CAT534C 3 passes	–	55
3	Vibratory Steel Roller CAT534C 6 passes	–	–	35

The results of the air void distributions for the gyratory compacted samples were similar to previous studies. The air void at the top and the bottom of the samples was higher than in the middle. This pattern was consistent for all compaction variable combinations, as can be seen in Fig. 3.15a. However, the non-uniformity in the air void distribution was reduced by increasing the angle of gyration and reducing the compaction pressure. The horizontal distribution of the air voids was also discussed; the results showed that the sample height was the main factor affecting the air void distribution in that direction. For the 135 mm height samples, the analysis revealed more air voids in the outer region closer to the mold, while the 50 mm showed an opposite trend. Figure 3.16 shows the air void distribution for compaction using two mold temperatures. A more uniform distribution of the air voids was realized at higher mold temperatures.

The different patterns for field compaction did not influence the air void distribution as shown in Fig. 3.15b. Field cores always had a high air void percentage at the top and decreased with depth down to one third of the sample. After that point, a more uniform distribution was observed. The results also showed that the horizontal air void distribution was similar for the different compaction patterns. The analysis showed that aggregate orientation was not affected by field compaction patterns. However, for the gyratory compacted samples, both gyration angle and the height of the sample affected the aggregate orientation. Figure 3.17 illustrates the aggregate orientation results.

The results of Tashman's study indicated that the adequate representation of field cores is accomplished when gyratory samples are compacted short (50 and

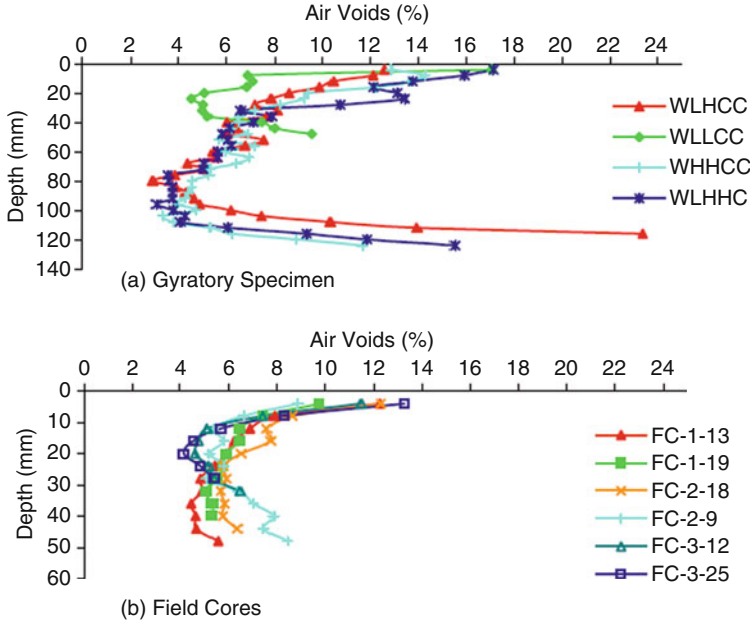


Fig. 3.15 Vertical distribution of air voids in gyratory specimens and field cores (Tashman et al. [32])

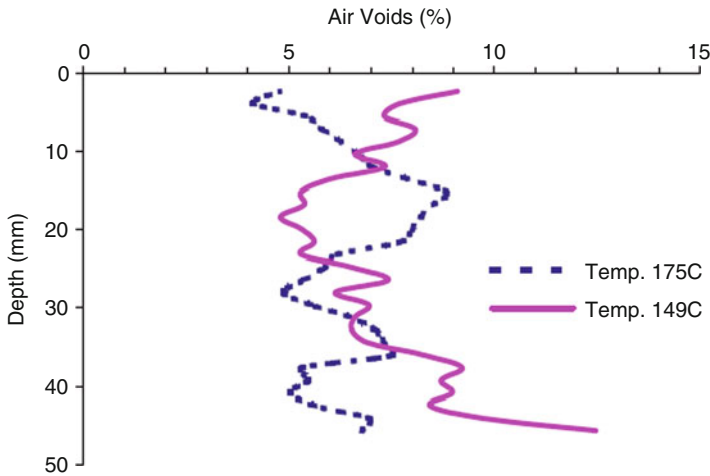


Fig. 3.16 Effect of the base plates and mold temperature on the vertical distribution of air voids in gyratory specimens (Tashman et al. [32])

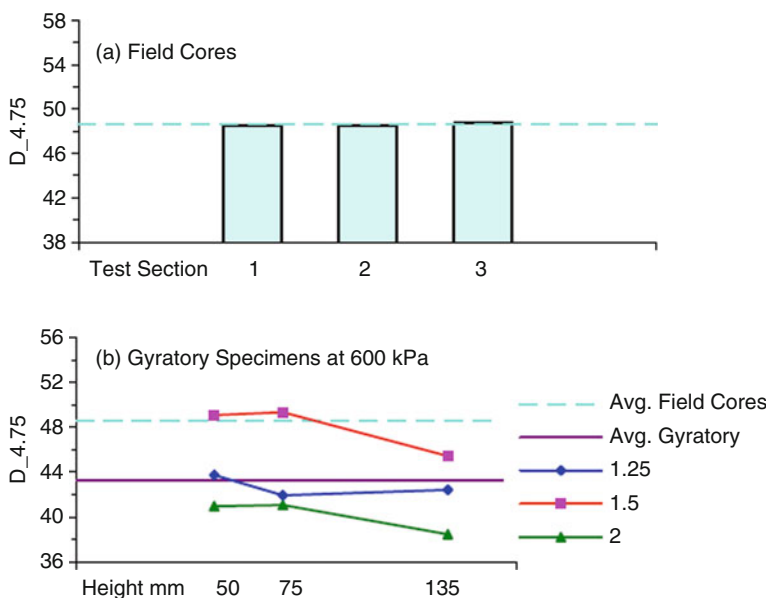


Fig. 3.17 Vector magnitude of aggregate orientation (Tashman et al. [32])

75 mm) and at angle of gyration of 1.5° . The aggregate contacts was also studied, the results showed that the aggregate contact distribution is random in both field and laboratory samples, and that there was no statistically significant difference between them.

3.4.4 Homogeneity of Laboratory Mixture Samples

The studies by Masad and Tashman focused the attention of researchers on the important issue of the heterogeneity of aggregate structures in mixtures and looking at the effect of compaction methods in laboratory. Partl et al. [28, 29] conducted significant work examining the homogeneity and isotropy of asphalt mixtures compacted using the compaction techniques of the Superpave gyratory compactor, the Marshall compactor, and the French roller compactor with pneumatic rolling-wheel.

The Superpave gyratory and the Marshall samples were cored from the center. The gyratory samples were then cut into four parts; top, upper middle, lower middle, and bottom. The Marshall samples were cut into three parts; top, middle, and bottom. Figure 3.18 illustrates the cutting of the rolling-wheel compacted samples.

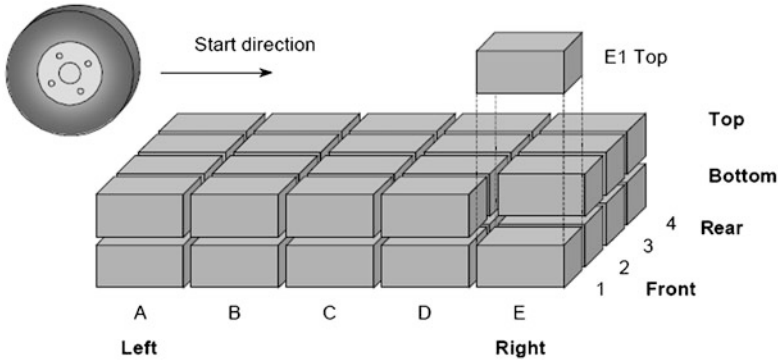


Fig. 3.18 Specimens compacted with French roller compactor using pneumatic rolling-wheel (Partl et al. [29])

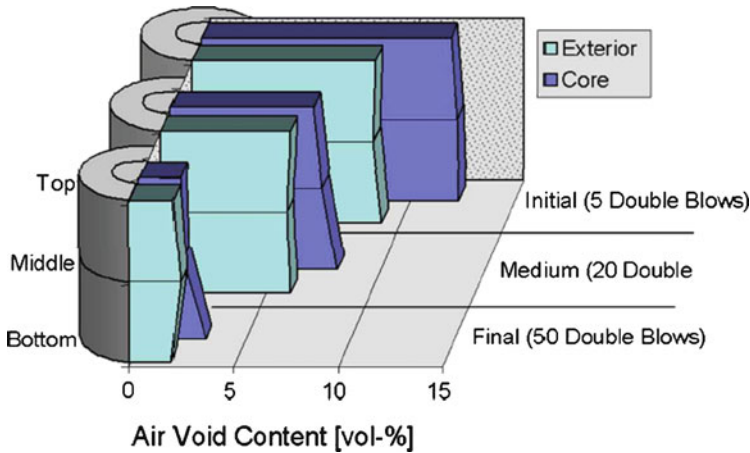


Fig. 3.19 Air void distribution in Marshall specimens (Partl et al. [29])

As shown, the samples were cut into 40 pieces. CT scans were used to investigate the air void distribution for the different samples. The following is a summary of the most important findings:

- For the Marshall compaction, there appeared to be a difference in the air void distribution between the core and the exterior parts of the samples, as shown in Fig. 3.19. This difference varied with the compaction effort. It was more pronounced at the initial stages and decreased at later stages. At 50 blows, the difference was also a function of depth and the bottom of this sample showed higher variation.
- For the Superpave gyratory compacted samples, the air void content results are summarized in Fig. 3.20. The middle of the cored samples was more compacted

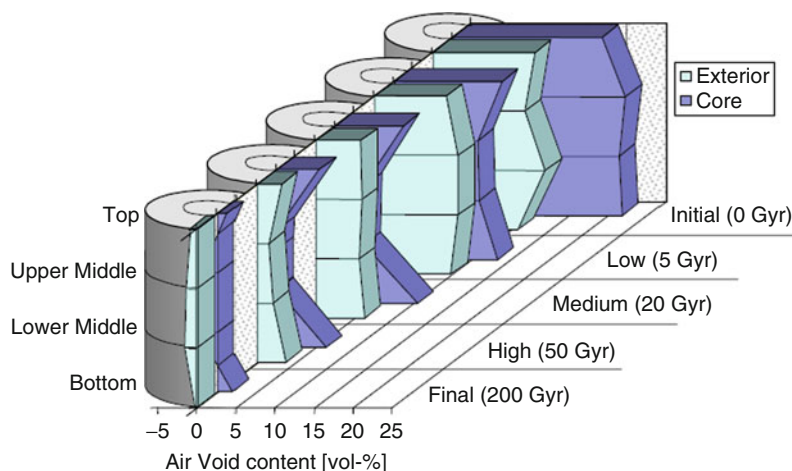


Fig. 3.20 Air void distribution in gyratory compacted specimens (Partl et al. [28])

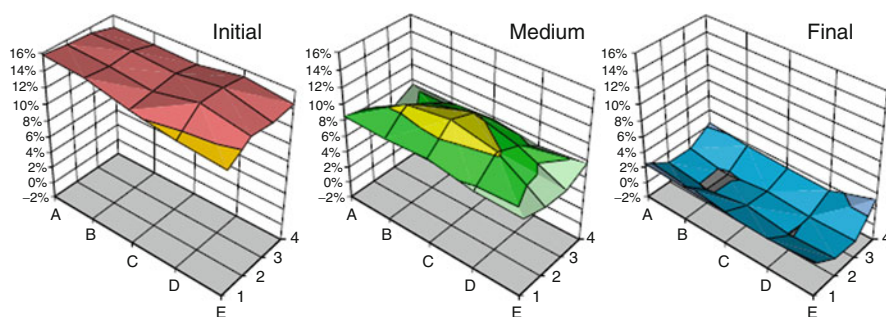


Fig. 3.21 Air void content in the *top part* of specimens compacted with French roller compactor using pneumatic rolling-wheel (Partl et al. [29])

than the top and bottom. However, the sample became more uniform as the number of gyrations increased.

- For the pneumatic rolling-wheel compacted samples, the air void content results are summarized in Figs. 3.21 and 3.22. The air void content distribution was almost uniform after the initial compaction. However, with further compaction, the distribution became non-uniform. Minimum air void content was observed at the center-bottom. After the final compaction, it was clear that the top was more compacted than the bottom, as the air void content values indicate.
- Comparing the three compaction methods revealed that the relationship between the air void content and the compaction effort was not the same for different compaction methods. Although the air void content decreased with more compaction effort for all methods, the reduction in the air void content followed a linear relationship for the Marshall compactor, non-linear for the Superpave

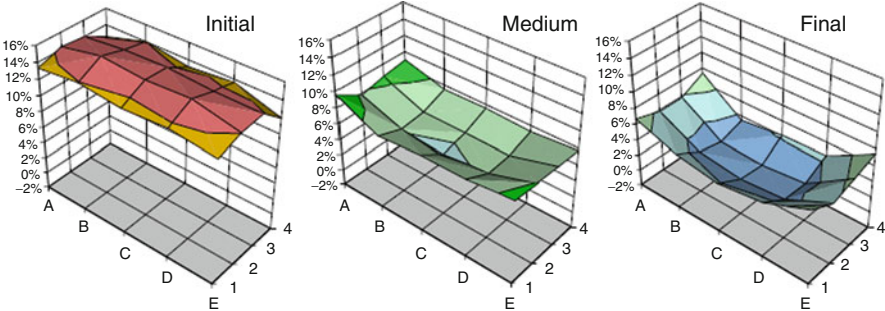


Fig. 3.22 Air void content in the *bottom part* of specimens compacted with French roller compactor using pneumatic rolling-wheel (Partl et al. [29])

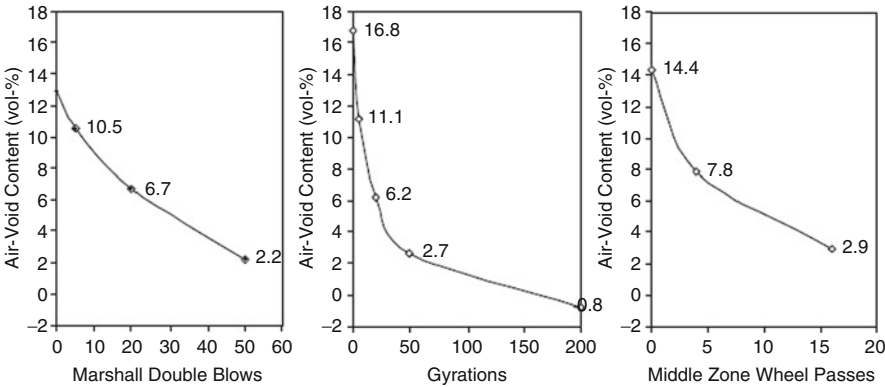


Fig. 3.23 Decrease in air voids content with the compaction level for 3 types of laboratory compactors (Partl et al. [29])

gyratory compactor, and intermediate state between linear and non-linear for the rolling-wheel compactor (middle zone of specimen), as shown in Fig. 3.23.

Partl’s extensive work was not the only effort in this field. Hunter et al. [17] also studied the effect of three laboratory compaction methods: gyratory, vibratory, and slab on internal structure. Ten samples were produced by each of the compaction methods. All the samples were prepared for the same mix design. The gyratory samples were compacted under 600 kPa pressure at a 1.25° gyration angle. Vibratory samples were prepared by applying a vibratory Kango hammer to the two sides of the samples. The slab compactor produced a 300 × 300 × 100 mm slab from which two 150 mm samples were then cored.

The internal structure was identified in this study by measuring the aggregate orientation and segregation. The orientation results showed that the orientation angle increased (toward 90°) with an increase in the aggregate size for the gyratory and vibratory compacted samples. This indicates that the larger aggregates favored

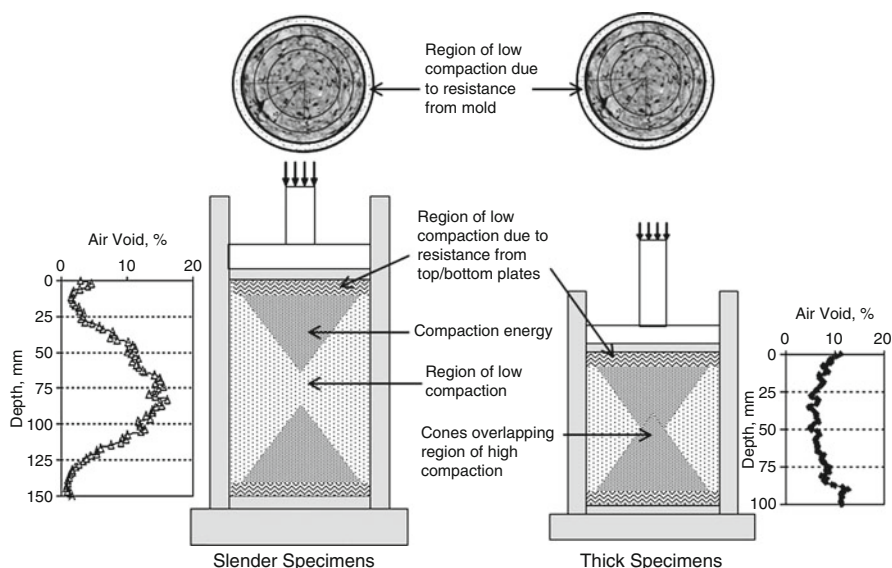


Fig. 3.24 Compaction energy distribution for asphalt samples with different heights (Thyagarajan et al. [33])

the circumferential alignment. The slab-compacted samples showed less increment for the circumferential alignment in general. The effect of sample depth and elongation of the particles on the aggregate orientation were studied as well. Both the gyratory and vibratory samples showed a greater orientation angle that also increased for higher aggregate size for elongated aggregate particles. However, depth did not significantly change the orientation angle.

Two types of aggregate segregation were evaluated in this study: radial and regional segregation. The radial segregation analysis indicated that the slab-compacted samples showed less radial segregation than the gyratory and vibratory compacted samples. Similar results were realized for regional segregation, where the slab-compacted samples showed almost a uniform particle distribution, while the vibratory compacted samples exhibited the largest regional segregation followed by the gyratory samples.

Thyagarajan et al. [33] studied the effect of different Superpave gyratory compactors and sample preparation conditions (cutting, coring, and sample height) on air void distribution and heterogeneity. The study was based on the fact that compaction energy is not uniformly distributed and the hypothesis that it follows a cone-shaped model. Figure 3.24 illustrates the energy distribution hypothesis and how compacting to different sample heights was expected to have different air void distributions. The main results of this study indicated that, in general, the vertical air voids distribution was not uniform. It was also found that the gyratory type sample height and sample preparation (cut/core) were significant factors that influenced the vertical voids distribution. Cutting and coring reduced the voids variability significantly in the vertical direction for all gyratory compactor types.

The results also showed that the lateral air void distribution was not uniform for all the gyratory compactor types. Air void content was the highest at the circumference (contact point with the mold) and decreased toward the center of the sample. It was shown that the lateral heterogeneity increased as the sample height decreased, which was the opposite of the vertical heterogeneity case. The cutting/coring process effect on lateral voids distribution was not consistent and the values of lateral voids variability were much higher than the vertical variability.

3.5 RILEM TG2 Group Study on Internal Aggregate Structure of Hot Mix Asphalt

As discussed, the development of a number of laboratory compaction methods was motivated by a common goal: to most accurately represent or simulate field compaction. Although each of the equipment developed claimed to more accurately reproduce field conditions, there is no agreement on a single best method. A number of studies have utilized imaging techniques. Details of how imaging has provided insight into internal parameters have been previously discussed. These internal parameters were air voids content and the distribution of voids, as well as aggregate orientation.

Imaging studies have used image-analysis software that varied in the details of defining aggregates, air voids, and binder domains in asphalt mixtures. The studies also varied in what defines aggregate structure and how to determine the internal parameters. The RILEM TG2 group study was initiated to address the standardization of image analysis for asphalt mixtures and define internal parameters.

3.5.1 Development of Analysis Software and Additional Internal Parameters

The study by the RILEM group focused on development of a new software capable of providing additional internal structure parameters: aggregate-to-aggregate contact points, aggregate orientation distribution and aggregate size spatial distribution and segregation. This software was used to analyze mixtures produced by compaction equipment still in use to determine how they influence the internal structure of a given mixture that has also been compacted in the field. The conceptual motivation of this work was that numerous studies showed that mechanical properties are affected by compaction methods, but have not shown why and how. It was hypothesized that measuring and defining internal structure could explain the reason for variations in performance. Although limited 3D images were included in the study, the focus was on using 2D images to quantify the internal aggregate structure in terms of aggregate-to-aggregate contact points, orientation, and segregation.

3.5.2 *Experimental Study*

A major part of the study focused on the Superpave gyratory compactor (SGC) method of compaction as it was the most rapidly spreading method in North America at the time. To ensure that the imaging technique developed could detect the effect of varying compaction conditions and equipment, additional methods and equipment including the Hveem, Marshall, and German steel sector were also studied. Images were all processed and analyzed with the same software developed and used in the study.

3.5.2.1 Participating Laboratories

In total, 13 labs from eight countries participated in this study. Loose mixture materials for compaction were provided by the former Laboratoire Central des Ponts et Chaussées, France (now IFSTTAR), and all other laboratories voluntarily spent time to compact, analyze data, or provide feedback throughout the process. The participating labs were Michigan Technological University, USA, responsible for Superpave Gyratory specimens; Technische Universität Braunschweig, Germany, responsible for German steel sector specimens; Asian Institute of Technology, Thailand, for provision of specimens compacted using the Marshall hammer; University of California Pavement Research Center, USA, for specimens compacted using the California kneading compactor; and Michigan State University and University of Wisconsin-Madison, USA, for extensive work in developing the software used in this study and the analysis of collected data. Additional laboratories participating in the activities of this study were Federal University of Ceará & Petrobras, Brazil; University of Palermo and University of Parma, Italy; EMPA – Swiss Federal Laboratories for Materials Testing and Research, Switzerland; University of Liverpool and University of Nottingham, United Kingdom; and Federal Highway Administration (FHWA), USA. With the help of the aforementioned laboratories, detailed evaluation of the software and effects of compaction conditions were conducted and the results and conclusions are presented herein.

3.5.2.2 Materials and Testing Conditions

The software, after careful testing and evaluation, was used for two main experimental studies to evaluate the effect of compaction methods and conditions in the lab on aggregate structure parameters. In the first study, images obtained from compaction of one mix design at different labs by the German steel sector, Marshall and Hveem compaction equipment were analyzed. The mixture was produced in the

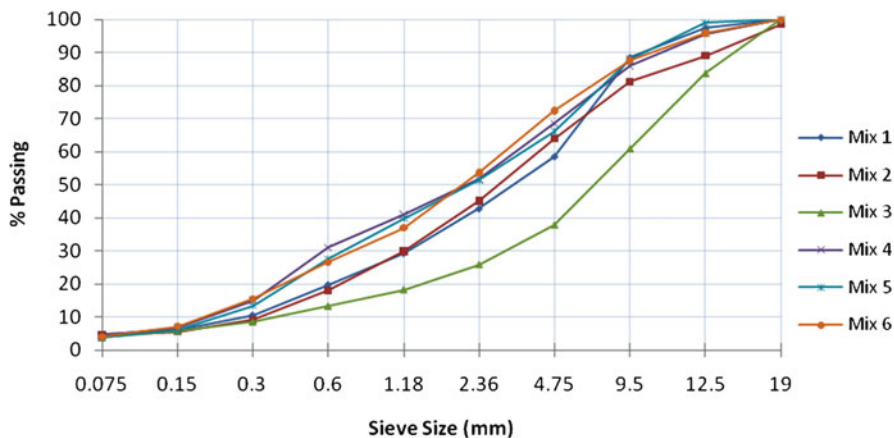


Fig. 3.25 Gradations of gyratory compacted specimens imaged

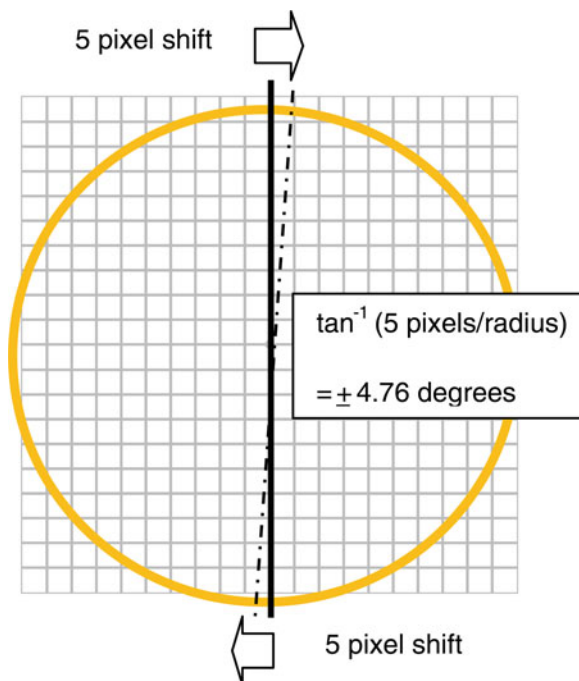
field by LCPC in France and was used for construction of a field testing section. The different labs shipped the samples for cutting, scanning and use of the software.

In the second study, the effect of temperature and pressure on samples compacted using the US gyratory compactor were studied. The samples included six mix designs that covered two nominal maximum aggregate sizes (NMAS), two gradations (fine and coarse), and two aggregate types (limestone and gravel). Figure 3.25 depicts the gradations used. For each combination, the mixtures were compacted at 60 and 120 °C paired with two compaction pressures of 300 and 600 kPa, yielding four combinations of temperature and pressure for each mix design. This level of variation in both temperature and pressure was expected to result in a clear distinction between aggregate structure measurements for the specimens.

3.5.2.3 Imaging Parameters

Images processed and analyzed in each study were captured at a resolution of 600 dpi (approx. 24 pixels/mm), which is the equivalent of approximately 0.04 mm/pixel. The distinction between sand and gravel is generally taken to be at 4.75 mm in diameter. At a resolution of 0.04 mm/pixel, the minimum aggregate or gravel identified by a diameter of 4.75 mm is seen in the image as approximately 120 pixels, or a radius of roughly 60 pixels. Using the default median filter size of five pixels, the software identifies the intensity of each pixel and the surrounding radius of five pixels, replacing each pixel with the median value of the five pixel radius region. This applies a “smoothing” effect to the image, which results in a minute loss of information in this processing step. The loss has the potential to

Fig. 3.26 Tolerance of orientation calculation



influence a zone of up to five pixels. Based on a five pixel shift of a diametrical line rotated about the center, the minimum aggregate size (radius of 60 pixels) can shift $\pm 5^\circ$, or a total swing/rotation of 10° . This is illustrated in Fig. 3.26.

This tolerance is critical when determining the orientation of individual aggregates relative to the major axis of the aggregate. The major axis has a tolerance of nearly 10° . For this reason, the analysis performed in the two studies determined frequency of angular occurrence and also a harmonic function fit to aggregate orientation presented in histogram form divided by increments of 10° (as finer increments would exceed the tolerance).

Although the software developed in this study is capable of measuring both horizontal and radial orientation, when comparing horizontal slices of laboratory compacted specimens from various methods, it is not reasonable to look at horizontal orientation. Orientation relative to horizontal requires a fixed point of reference and is more appropriate for field cores where the standard reference direction is assigned as either the longitudinal or transverse direction of the roadway. For laboratory compacted specimens, this standard reference no longer applies due to the varying geometries of compaction equipment and the ease of freely rotating the specimen after compaction. For this reason, the effect of compaction method on aggregate orientation was determined by comparing radial orientation (angle θ measured between aggregate major axis and radial arm drawn from center of image to center of aggregate), and is presented herein. Figure 3.27a shows a standard histogram produced by dividing an individual

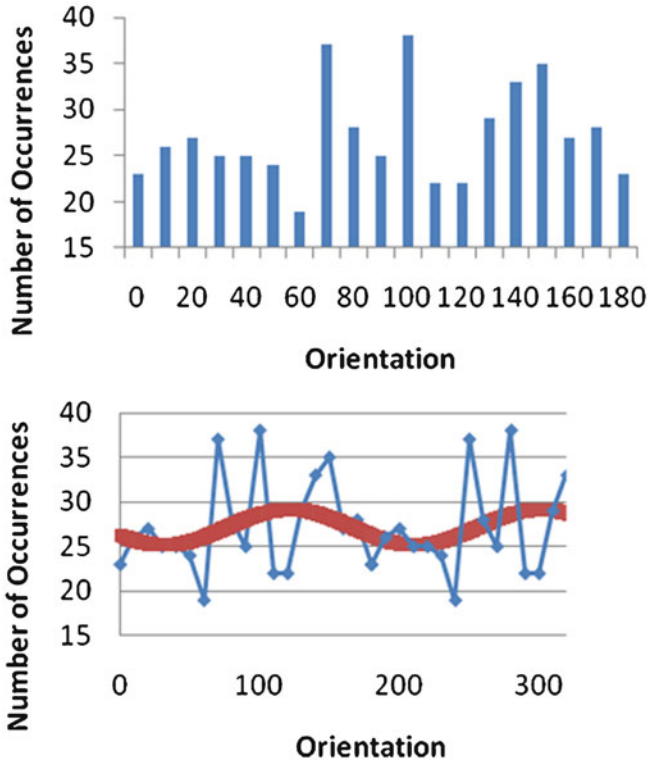


Fig. 3.27 Conversion from (a) histogram representation to (b) harmonic fit

aggregate orientation into 10° intervals. Fig. 3.27b shows the same frequency of occurrence per angle represented as a line graph rather than a bar chart with the addition of a harmonic fit.

Tashman et al. [32] presented the idea of assigning a harmonic function to be used as a fit for the data. This fit clearly identifies (by the maximum of the sinusoidal wave) the predominant aggregate orientation within the image. This is accomplished by using the following equations:

$$\text{freq}_{\text{harm}} = \text{freq}_{\text{ave}}(1 + a^* \cos 2\theta + 2b^* \sin \theta \cos \theta - a^* \sin 2\theta) \quad (3.9)$$

where,

$$a = \frac{2 \sum_{k=1}^N \cos(2\theta_k)}{N} \quad (3.10)$$

$$b = \frac{2 \sum_{k=1}^N \sin(2\theta_k)}{N} \quad (3.11)$$

and N = total number of aggregates accounted for in image.

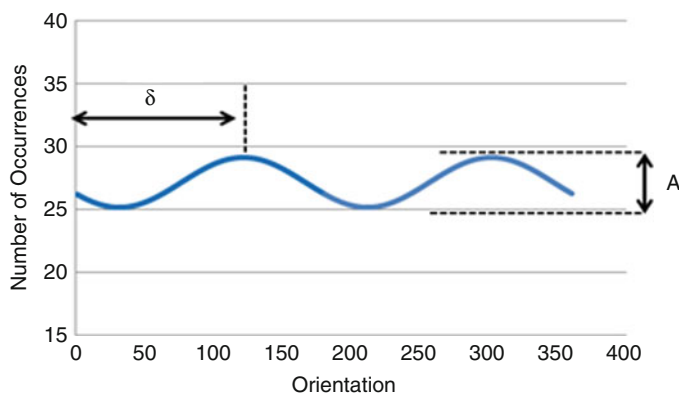


Fig. 3.28 Additional parameters defined for harmonic fit

Since the fit used is a harmonic function, localized maxima (180° apart) and the amplitude of the function can be identified as shown in Fig. 3.28. The δ identifies the predominant orientation as determined by the fit, whereas the amplitude (A) of the harmonic is a measure of the severity or variance from a uniform distribution. A uniform distribution of aggregate orientation would result in an amplitude of zero, i.e. a horizontal line. Thus, the lower the amplitude, the less severe is the reported non-uniformity of orientation.

3.5.3 Results and Analysis

In comparing the effects of various compaction methods on aggregate structure using the technique described, harmonic fit was used, as shown in Fig. 3.29 and Table 3.3. The results indicate that there are significant effects of the compaction method on the calculated parameters (A and δ). For the analyzed mixture, the Superpave gyratory compactor had the greatest value for A (7.3), which implies that it has the highest tendency to orient the aggregate to a particular angle, 83° from the radial arm in this case. The Marshall had the lowest severity (value of A), or closest to uniform, of the four methods with an amplitude of 2.78.

The relative ranking of compaction method according to severity of aggregate orientation places Marshall as the least influential and Superpave Gyratory as the most influential, with the Hveem and German steel sector falling in between. The Superpave gyratory also reported a predominant angle closest to 90° , which is indicative of aligning the aggregates concentric to the perimeter. This is expected since the gyratory is a circular compaction method.

To further study the effect of compaction conditions (temperature and pressure), the standard Superpave gyratory compaction technique was used for the second study presented herein. The temperature and pressure applied via the Superpave

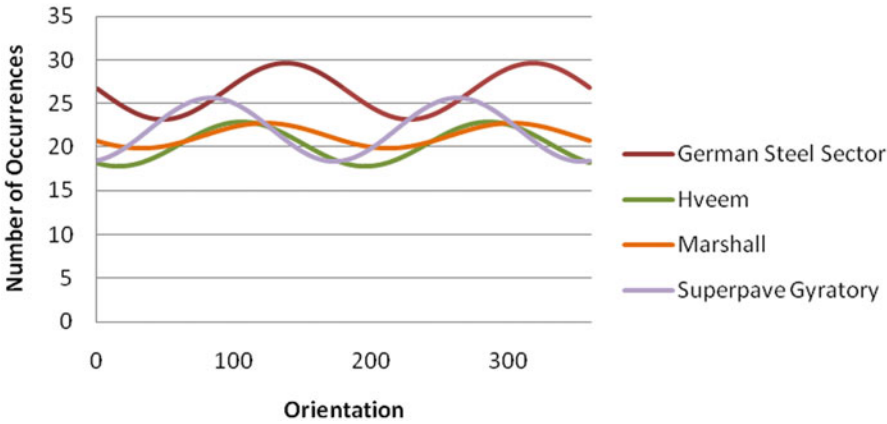


Fig. 3.29 Effect of compaction method

Table 3.3 Predominant angle (δ) and severity (A) of various compaction methods

Compaction method	δ	A
Hveem	106	5.14
Marshall	122	2.78
German steel sector	138	6.38
Superpave gyratory	83	7.3

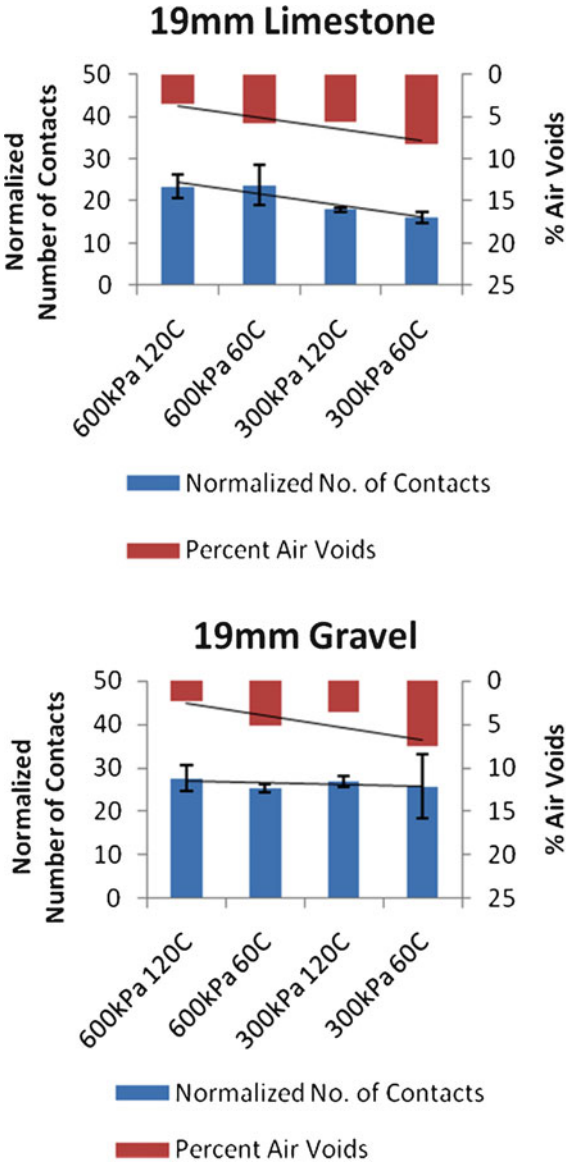
gyratory compaction equipment was varied to test how this influences the physical properties of the mixtures as identified by imaging.

3.5.3.1 Effects of Compaction Temperature and Pressure in the Gyratory Compactor

Several combinations of compaction temperatures and pressures were used in this secondary study to determine the effects of varying compaction conditions on the aggregate structure parameter. The image analysis compared the severity of aggregate orientation as well as the number of contact points between adjacent aggregates in each image, as shown in Figs. 3.30 and 3.31. It is seen in these figures that, for the variety of specimens chosen (which include 19 and 12.5 mm NMAS, limestone and gravel aggregates), the number of contact points found in the image follows a similar trend to that of percentage of air voids found in the mixture using conventional laboratory techniques.

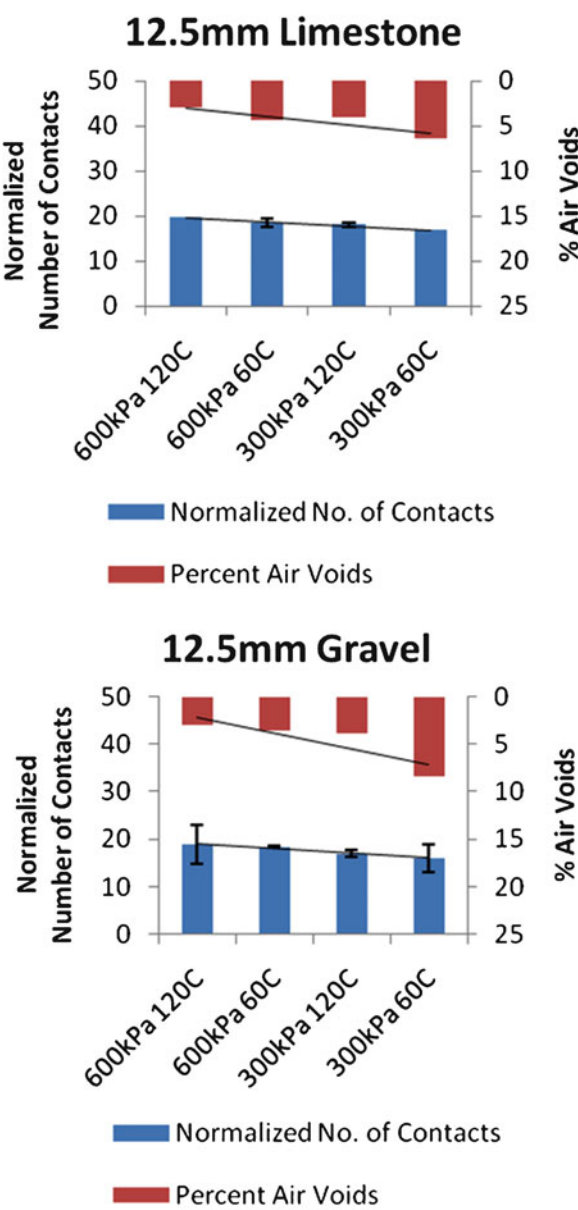
To avoid a false influence of gradation of each aggregate type and nominal maximum aggregate size, the number of contact points identified in each image was normalized by the gradation (10% of the percent retained on the 4.75 mm sieve). The collective results, although showing variation, follow a trend indicating that a

Fig. 3.30 Comparison of normalized number of contact points and percentage of air voids for 19 mm mixtures



higher void content results in fewer contact points. In other words, lower temperature and lower pressure lead to fewer contact points. In a sense, this small data set confirms the current practice where density is a critical indicator of asphalt mixture stability since it leads to higher contact points. Obviously, changes in gradation for a given density could result in a higher or lower number of contact points. If a simple tool for direct estimate of contact points is developed, it would be a critical complement to density for reducing risk of mixture instability. The results shown in

Fig. 3.31 Comparison of normalized number of contact points and percentage of air voids for 12.5 mm mixtures



Figs. 3.30 and 3.31 for a normalized number of contact points represents the average value as determined by two images, with the error bars indicating the high and low of the two images.

Table 3.4 Orientation parameters of 19 mm gyratory specimens

Compaction temperature and pressure	δ	A
19 mm gravel		
600 kPa & 120 °C	78	20.46
600 kPa & 60 °C	110	5.35
300 kPa & 120 °C	86	10.02
300 kPa & 60 °C	111	11.15
19 mm limestone		
600 kPa & 120 °C	110	15.21
600 kPa & 60 °C	122	15.13
300 kPa & 120 °C	94	21.08
300 kPa & 60 °C	88	10.96

Table 3.5 Orientation parameters of 12.5 mm gyratory specimens

Compaction temperature and pressure	δ	A
12.5 mm gravel		
600 kPa & 120 °C	83	21.34
600 kPa & 60 °C	67	15.78
300 kPa & 120 °C	104	7.22
300 kPa & 60 °C	102	15.17
12.5 mm limestone		
600 kPa & 120 °C	73	6.42
600 kPa & 60 °C	91	5.42
300 kPa & 120 °C	71	27.03
300 kPa & 60 °C	104	13.88

3.5.3.2 Analysis of Orientation Results

The orientation of the aggregates within each gyratory specimen was evaluated on the same bases as those used for comparison of different compaction methods; using the parameters of predominant angle of aggregate orientation (δ), and the level of severity (A) estimated from the harmonic fit of the orientation spectrum. This evaluation is tabulated in Tables 3.4 and 3.5. No clear trend was identified in either the predominant orientation angle, δ , or the severity, A, in either of the tables. The tabulated values are also illustrated in Fig. 3.32. The dashed lines in the figure represent the median orientation and median severity, respectively. No discernible trend can be seen. This indicates that, although the Superpave gyratory compactor appears to result in the highest amplitude, A, of harmonic (which indicates a very narrow distribution of aggregate orientation values), changing the compaction temperature or pressure does not appear to significantly influence the amplitude. It is thus believed that the type of compaction is more important than temperature or pressure.

3.5.3.3 Effect of Nominal Maximum Aggregate Size (NMAS) and Gradation

This section discusses the effect of NMAS and aggregate gradation on the number of contact points. Initial analysis of various NMAS showed that a larger NMAS

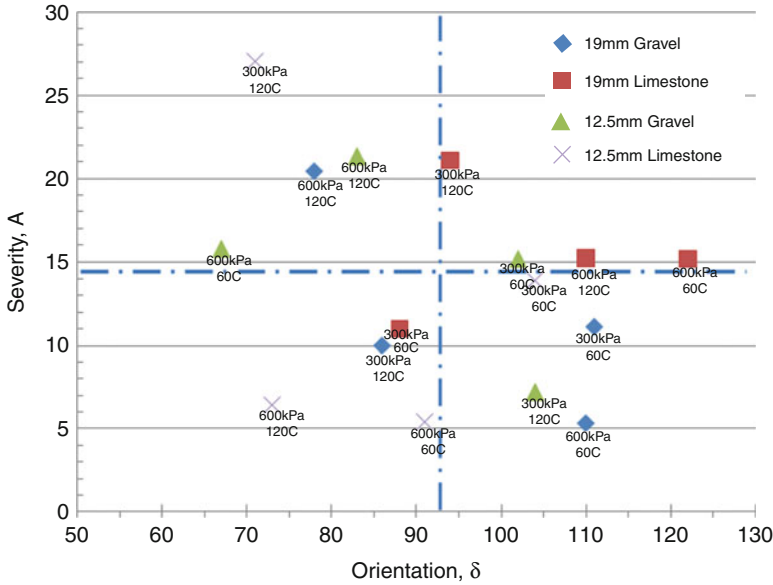


Fig. 3.32 Influence of compaction temperature and pressure on orientation parameters δ and A

consistently resulted in a greater number of contact points identified in the image. To better account for the effect of gradation on number of contact points, the individual particle size distribution of each mixture must be more closely examined. To do so, the Weibull cumulative distribution function was used to fit each mixture gradation used in this second study:

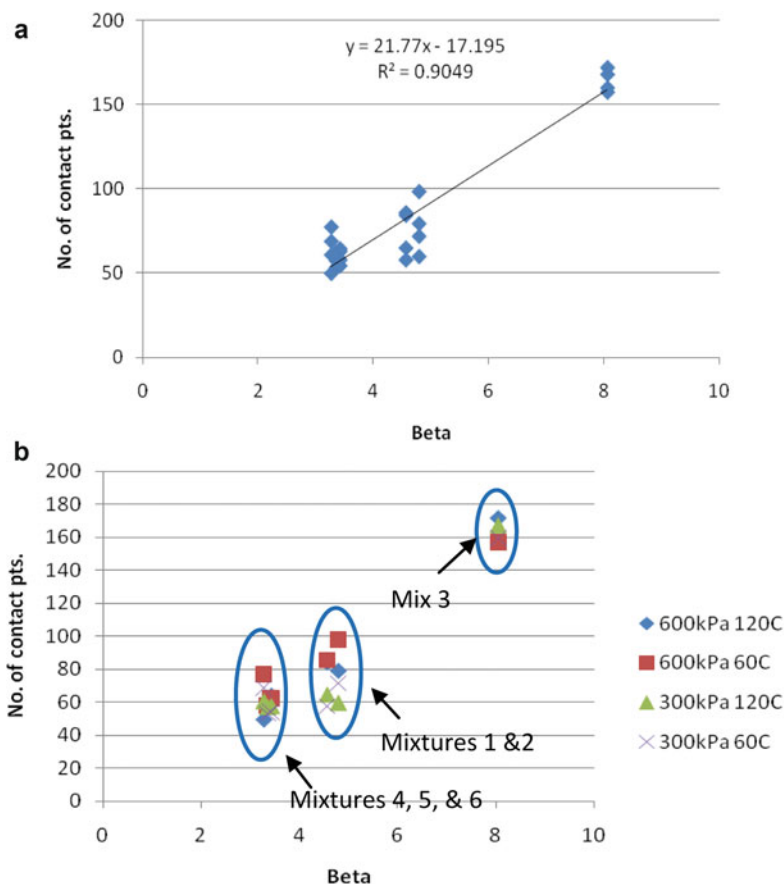
$$F(x, \alpha, \beta) = 1 - e^{-(x/\beta)^\alpha} \quad (3.12)$$

where β indicates shape of the gradation (fineness/coarseness) and α is a position parameter indicating how dense the mixture is (Mahmoud and Bahia [21]). F is the cumulative percent passing and x is the sieve size.

The corresponding α and β values for the six mixtures compacted in this second study can be seen in Table 3.6. The β parameter was correlated with the number of contact points, as shown in Fig. 3.33a, which shows a strong correlation between β and the number of contact points. The trend shown allows for the separation of mixtures based on specific gradation. As seen in Fig. 3.33b it is clear that, for all combinations of temperature and pressure, the data points for an individual gradation are closely grouped together. This grouping of mixtures according to gradation is in agreement with the Weibull parameters in Table 3.6, where β values are approximately 3.3–3.4, 4.6–4.8, and 8.1. This suggests that gradation, as defined by β , has a significant influence on the number of contact points identified within an image. It also shows that, although there could be some effect of pressure or temperature, the gradation type (fine or coarse) as defined by the β values has a much higher influence on the contact points.

Table 3.6 Alpha and beta values determined by Weibull distribution fit for each gradation

Gradation	Alpha	Beta
Mix 1	0.883	4.792
Mix 2	0.802	4.564
Mix 3	0.993	8.059
Mix 4	0.712	3.354
Mix 5	0.760	3.422
Mix 6	0.762	3.269

**Fig. 3.33** Beta value vs. number of contact points for (a) all gradations of study and (b) separated by compaction temperature and pressure

This finding is logical and expected. The finding also supports the concept that contact points could be a better indicator of quality and consistency of asphalt mixtures than density. There is no question that mechanical properties are affected by contact points. The critical question is, however, what the level of this effect is and whether contact points can be good predictors of stiffness of mixtures.

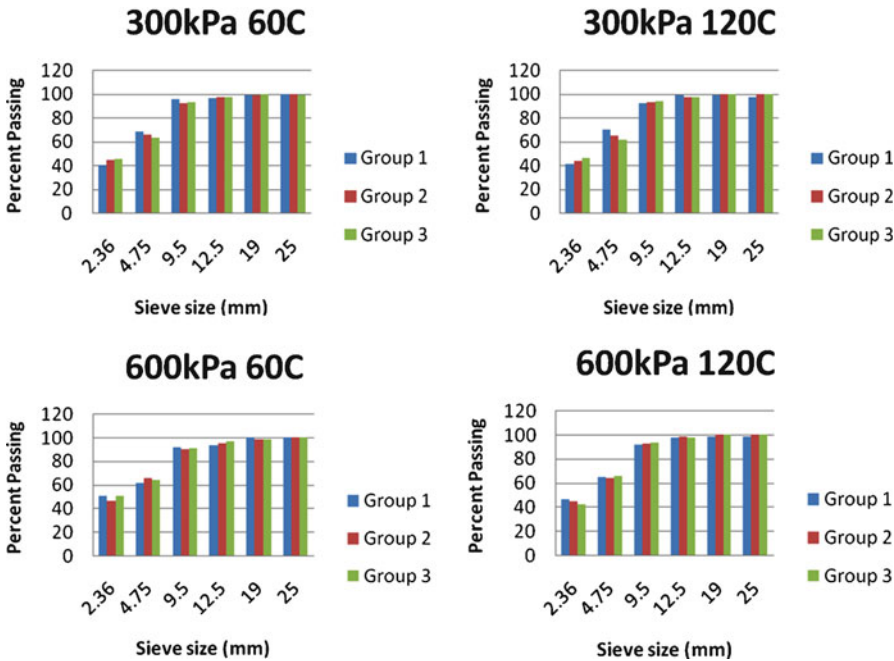


Fig. 3.34 Grain-size distribution of radial groups for 19 mm limestone mixture used to determine level of segregation

3.5.3.4 Segregation

The third analysis tool of the software developed throughout the work presented here is determining the segregation of aggregate within a mixture. Segregation provides further insight as to whether or not the gyratory action has any influence on aggregate migration within the mixture relative to particle size. To investigate this, the percent passing each sieve size from one standard sieve above NMA to one sieve size below 4.75 mm (the particle size used for contact points and orientation evaluation) was determined for each mixture. An example of the results of these partial gradations are illustrated in Fig. 3.34 for the 19 mm limestone. The results show no evidence of segregation. In fact, none of the mixture images analyzed in the second study displayed extreme segregation of aggregates caused by using the gyratory compactor. Although the study did not show variation in spatial distribution, it is believed that segregation remains a very effective function of the software that can be used in further investigation of the internal structure of HMA.

Table 3.7 Predominant angle (δ) and severity (A) from field samples

Location	δr_4 mm	A
F10 left	94	3.46
F10 center	97	4.14
F10 right	69	4.5
F20 left	71	1.06
F20 center	88	0.6
F20 right	102	2.54
Average	86.83	2.72

3.5.3.5 Comparison to Field Compaction

It should be restated that, although the software developed here determines aggregate orientation relative to two bases (horizontal and radial) when comparing the various laboratory methods (first study), many different geometries are encountered. Due to variation in geometry as well as shipping from around the globe, all laboratory compacted specimens were cored to 100 mm diameter and collected at a central location. Once cored and shipped, any pre-determined compaction direction specific to equipment no longer has a set reference, meaning the cylindrical samples were analyzed relative to the radial orientation rather than the horizontal. To better understand this selection, one can think of the scenario in the following manner; when measuring aggregate orientation relative to horizontal on the screen/image, the horizontal or vertical direction must have a specific meaning or coincide with a known direction in the lab or the field. Since this was not clearly identified as the compaction direction – longitudinal, or transverse for the laboratory specimens – the horizontal reference becomes arbitrary and is dependent on the user scanning the digital images of the physical specimens. Multiple users may result in multiple rotations of the sample (horizontal slices) on the scanner glass, ultimately changing the identified predominant aggregate orientation. To avoid this arbitrary assignment, the radial angle (angle from a radial arm drawn from center of image to center of aggregate relative to the major axis of the aggregate) was used. Any rotation of sample in shipping or scanning will not result in a change to this parameter.

With this in mind, although the longitudinal versus transverse direction can be clearly identified when analyzing images from the field (in this case a test track), to remain consistent with the analysis of laboratory methods and thus determine which laboratory compaction method most closely reproduces the internal aggregate structure observed in the field, the radial angle must again be used for the field cores. Table 3.3 showed the predominant angle (δ) and severity (A) identified by harmonic fits applied to the distribution of aggregate orientation found in each of the studied laboratory compaction methods. This can be compared to obtained field values as listed in Table 3.7.

F10 and F20 in Table 3.7 represent two locations on the LCPC test track. Since the samples taken were larger than typical laboratory samples, images were analyzed to take areas similar to the laboratory samples but at the left, center and right for each location. Figure 3.35 depicts the details of the location of the cored samples on the LCPC test track.

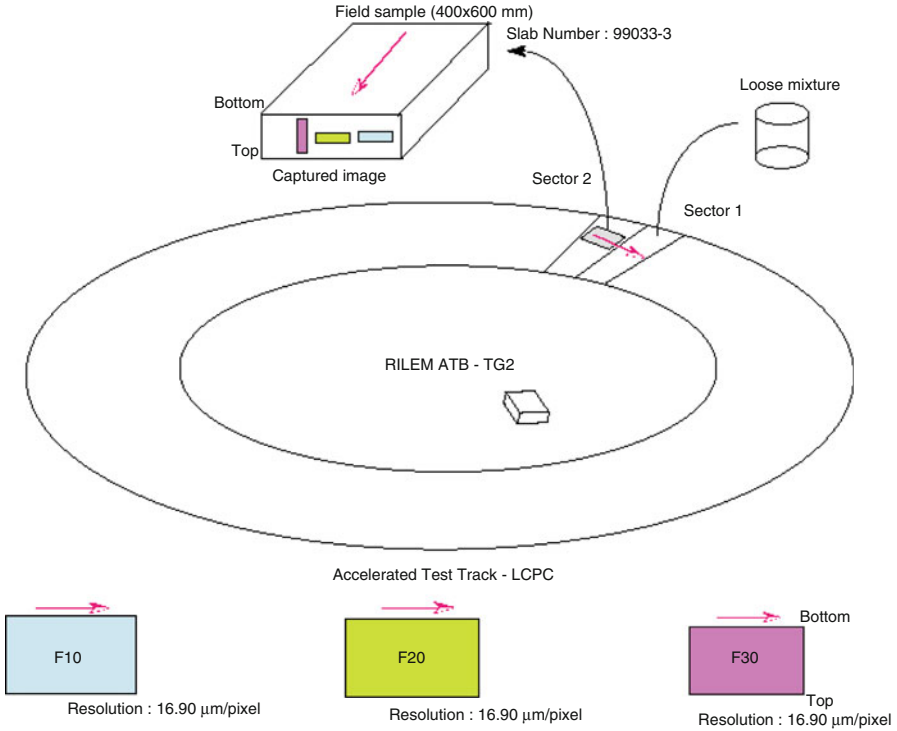


Fig. 3.35 Details of sampling locations on the LCPC test track

Comparing the average predominant angle of approximately 87° with the various laboratory compaction methods, it can be seen that this aggregate orientation is most closely reproduced by the gyratory compactor, although an average severity of 2.72 from the field relative to 7.3 produced by the gyratory suggests that the orientation of aggregate in the field is not as heavily influenced as by the gyratory and that the level of severity is best represented by the minimally influencing laboratory method of the Marshall hammer (a severity of 2.78).

To better understand which of these two compaction methods best simulated aggregate orientation in the field, we must revert to the harmonic fits derived from the individual data. For this reason, Fig. 3.36 shows the aggregate orientation histograms represented by the corresponding harmonic fit for the gyratory, Marshall and field samples.

Since a histogram is representative of the number of occurrences, or number of individual aggregates, that exhibit a given angle, it would be misleading to combine all three images at both locations from the field (for a total of six images), when each of the laboratory methods was represented by two imaged surfaces as depicted in Fig. 3.33b. This would allow the field images to be represented by 3x the analyzed area. To avoid this deceptive comparison, Fig. 3.36 shows the two locations, F10 and F20, individually represented after determining any outliers of

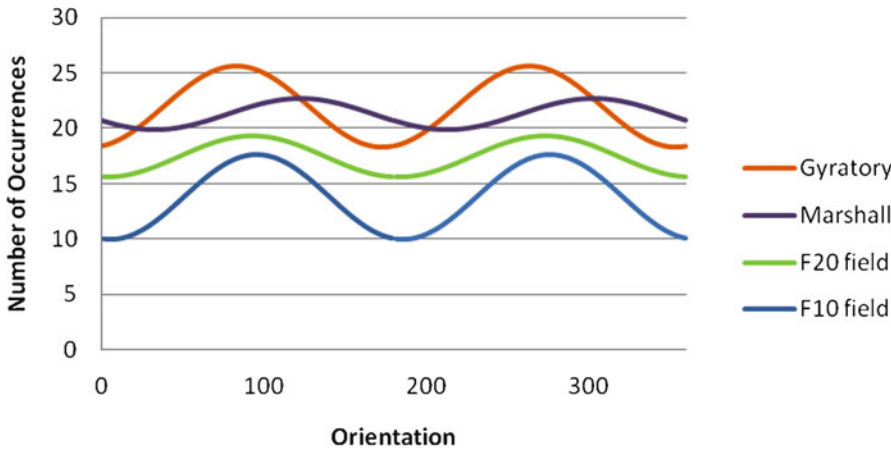


Fig. 3.36 Aggregate orientation histograms as *line graph* (based on bins of 10° as deter-mined by resolution of images)

Table 3.8 Combined predominant angle (δ) and severity (A) from field samples

Location	$\delta r_{4\text{ mm}}$	A
F10	96	7.60
F20	93	3.74

the three (left, center, and right) based on data presented in Table 3.7. The results of this additional analysis are presented in Table 3.8.

It can be seen from the data presented in Table 3.8 that the predominant aggregate orientation continues to be more similar to that of the gyratory samples. It can also be seen that the repeatability of orientation of aggregate at a given field location allows for a histogram representation to begin to show an amplified severity as peaks and valleys of individual harmonics combine for an exaggerated harmonic fit. This increased severity, specifically at location F10, also begins to approach that of the gyratory compactor, while location F20 still exhibits a slightly lower severity with a similar predominant aggregate orientation.

Although significant aggregate orientation distribution differences could be captured for the samples F10 and F20, further validation of these conclusions is needed. Replicates of field samples should be analyzed to find the spatial distribution of the predominant angle and severity in replicates taken from different locations. The results of such an analysis can be used to determine the statistical significance of the effect of loading in the wheel path on the observed differences between results of the F10 and F20 samples.

This topic was investigated for gyratory compacted asphalt samples, in which replicates of gyratory compacted samples with the same volumetric designs showed

no significant difference in terms of orientation distribution and aggregate structure connectivity (Coenen et al. [7]; Roohi et al. [31])

3.5.4 Summary of Findings and Conclusions of the RILEM Study

These studies focused on developing user friendly software for 2D image analysis. The all-inclusive software allows for one program to be used for both processing and analyzing a 2D image. The processing of the image prior to analysis ensures accuracy of results. This advanced technique was used on images of laboratory compacted asphalt concrete specimens that were captured directly from an affordable flatbed scanner available to the general public. The unique processing steps along with calculation of contact points between aggregates were verified by determining an appropriate surface distance threshold (SDT) that maximizes the accuracy of calculations while not exceeding the resolution of the image.

The analysis features of the software were used to compare mixtures produced using various compaction methods of laboratories in several countries. A method for representing orientation distribution by fitting a harmonic function indicated that there are distinct differences in aggregate orientation from various compaction methods. The Superpave gyratory showed the highest influence on aggregate orientation in a radial fashion. Additional investigation will be required to determine which of these laboratory methods most accurately reproduces the internal structure parameters seen in field compacted specimens.

Additionally, the second study focused on effects of various combinations of compaction temperatures and pressures used for the Superpave gyratory compactor to determine influence on aggregate structure indicators. It was found that the specific temperature and pressure of gyratory compaction did not significantly exaggerate or minimize aggregate orientation, although a clear difference in the number of contact points between aggregates was identified. The number of contact points within the gyratory specimens was found to directly correlate to the percentage of air voids in the mixture, which is commonly used to characterize flexible paving materials. A larger nominal maximum aggregate size was also found to increase the number of contact points within a given mixture.

A distinction was seen in the number of contact points relative to the aggregate type (gravel versus limestone). However, no real conclusions can be drawn as to whether this is image-clarity related or angularity related until further investigation of angularity is conducted for materials of this study.

To achieve the objective of standardization of image processing and analysis for asphalt mixtures, a draft standard following the guidelines of AASHTO was developed and sent for review by a number of experts who serve as members of the Federal Highway Administration Mixture Task Group in the US. Feedback was used to revise the standard. It is hoped that a similar standard will be developed in Europe. The software and the draft standard can be found on the University of Wisconsin Modified Asphalt Research Center web site (UWMARC.org).

3.6 Concluding Remarks

This part of the report covered the topic of asphalt mixture compaction methods and effects of compaction conditions on aggregate structure. The literature review conducted showed the potential benefits of advancing the techniques for imaging and analysis to understand the reason for variation in mixture mechanical properties due to compaction methods. It has also shown that imaging could provide another type of mixture characterization that is different than, and perhaps complementary to, the conventional volumetrics and mechanical characterizations commonly used today.

Visualization of aggregate structure using a simple digital scanner or a CT scan has been used during the last 10+ years to bring attention to the challenges faced today in matching the mixture aggregate structure of lab and field compacted samples. It has been also used to show the non-uniformity of voids and aggregate orientation within a sample. While visualization by itself can hardly replace the need for mechanical testing, developing a standard imaging and analysis tool could make visualization an important tool that mixture designers can use to better optimize asphalt mixture design, and possibly control its quality.

The software developed in this RILEM study was utilized by many participants and was found to be user friendly and effective. A draft standard document was reviewed by experts and revised based on their feedback. New parameters for defining aggregate structure in terms of orientation, special distribution, and contact points are proposed in the draft standard.

An initial study of one mixture was successfully concluded in which more than ten labs from eight countries have participated. Results show the software is useful in quantifying the variation of aggregate structure due to compaction method and conditions for a specific method. This initial study should encourage others to utilize this tool to advance understanding of asphalt mixtures and improve mixture design procedures in the future.

Authors of this section are very grateful for all the labs that participated in the study and also to Dr. Enad Mahmoud and Dr. Laith Tashman for their input and contribution throughout the study and writing of this report. Special thanks are also noted to the LCPC research team, in particular Dr. Chantal de La Roche for her significant support and contribution throughout the study.

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Chapter 4

Mechanical Testing of Bituminous Mixtures

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Cédric Sauzéat, and Didier Bodin

Abstract This chapter focuses on permanent deformation, which is at the origin of pavement rutting. First experimental laboratory tools and associated analyses related to permanent deformations are presented. It is either material tests, that can be homogeneous or giving a simple empirical characteristic, or non-homogeneous Structural/Specimen tests.

Then, investigations made, by Working Group 3 (WG3) “Mechanical Tests for Bituminous Materials” of the RILEM 206-ATB, are presented. These investigations consider only wheel Tracking Tester (WTT) devices used to evaluate the rutting performance of bituminous mixtures. A first study considers three different bituminous layer systems made with pure bitumen and currently used in road construction. A surprising obtained result is the very poor performance obtained on WTT for one system, while very good rutting resistance is observed on the road.

A second study is performed in order to evaluate the reproducibility of the French Wheel Tracking Test (FWTT) on a mix made with Polymer Modified Bitumen (PmB). For this type of mixture, some peculiar problems had been reported, such as heating of the sample due to friction and sticking of the binder to the wheel. Two procedures, called anti-overheating and anti-sticking procedures, have been proposed by the group in order to improve the reproducibility of the wheel tracking test. Causes of scattering are discussed. Differences between the

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tires used during this RRT as a possible origin of scattering are discussed, as well as temperature increase and over-compaction of specimens.

The last section presents modeling and simulation using Finite Elements Method performed by the RILEM group. The aim is to build numerical framework based on finite element (FE) simulations for rutting prediction of asphaltic layers. Before performing permanent (viscoplastic) deformations simulation, the work first concentrates on simulating the reversible viscoelastic stresses and strains during wheel tracking tests. The linear viscoelastic (LVE) mixture behavior is modeled by a generalized Kelvin-Voigt or generalized Maxwell model, with fixed and given parameters obtained from the continuum spectrum model 2S2P1D. Both generalized Kelvin-Voigt and Maxwell models, that are equivalent, are presented as well as their calibration. Then some FE simulations are described and their results are presented.

Keywords Asphaltic materials • Testing • Modelling • Thermo-mechanical properties • Permanent deformation • Rutting

4.1 Introduction

4.1.1 *Mechanical Properties of Asphaltic Mixtures*

In order to satisfactorily perform in pavement systems, asphaltic mixtures must exhibit: (a) the ability to distribute stresses; (b) resistance to permanent deformation; (c) resistance to cracking (especially fatigue cracking, but also thermal, reflective and construction cracking); (d) resistance to freeze-thaw and moisture damage.

The ability to meet these requirements is related to the fundamental mechanical properties of the mixture. These are affected by numerous factors related to the constituent materials (binders, aggregates, additives and their proportions), load (loading rate, loading and rest times, load level, stress state etc.), environmental conditions (temperature, moisture etc.) and construction (compaction method, quality control etc.).

There are three mechanical properties which are particularly important:

- Stiffness (in the linear domain)
- Resistance to fatigue cracking
- Resistance to permanent deformation

When considering the plot of Fig. 4.1, proposed by Di Benedetto [56] it is possible to identify the domains corresponding to the different aspects introduced previously and the corresponding “typical” type of behaviour for the mixtures. Of course, this figure should be considered as a general indication. The boundary of the different domains, shown on axes of logarithm of strain amplitude versus logarithm of the number of cycles, is an indication and the transition from one domain to another is made in a smooth and continuous way.

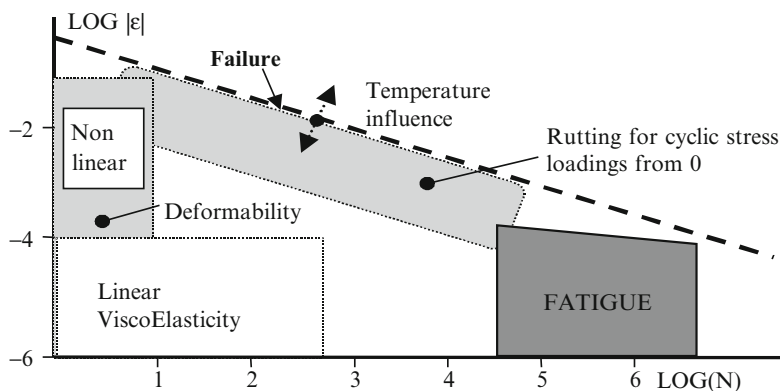


Fig. 4.1 Domains of behaviour for bituminous mixtures, (ϵ) strain amplitude – N number of cycles) by Di Benedetto [57]

Stiffness properties mainly characterised by the complex modulus, were studied by RILEM TC 101 BAT “Bitumen and Asphalt Testing” and TC 152 PBM “Performance of Bituminous Mixes”. Main results are presented in Di Benedetto and Francken [58] and Di Benedetto et al. [59].

Working Group 3 (WG3) “Mechanical Tests for Bituminous Materials” of RILEM TC 182 PEB “Performance and Evaluation of Bituminous materials” investigated the issue of fatigue from 1998. The work carried out by this WG is summarized in Di Benedetto et al. [60].

As a natural continuation of the previous work, WG3 of RILEM TC 206-ATB “Advanced Testing of Bituminous Materials”, decided to study, between 2006 and 2010, permanent deformation, which is at the origin of pavement rutting. This chapter presents a general development on permanent deformation and the main results obtained by the working group. All the participants to TG3 activities that are listed below are warmly thanked for their involvement and help to obtain key results on this phenomenon and make for a very agreeable and efficient common working atmosphere.

4.1.2 Type of Rutting on Road

Permanent deformation of asphaltic mixtures is one of the major mechanisms of pavement failure throughout the world.

There are two types of pavement rutting; namely structural rutting and non-structural rutting. Structural rutting involves deformation in the lower pavement layers (see Fig. 4.2). Non-structural rutting, on the other hand, is associated with permanent deformation in the asphaltic layers, which is usually accompanied with shoulders at the edge of the rut at the road surface as shown in Fig. 4.3.

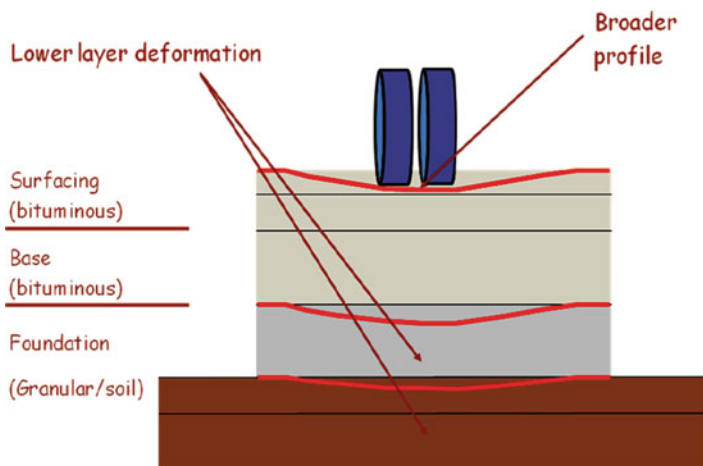


Fig. 4.2 Structural rutting

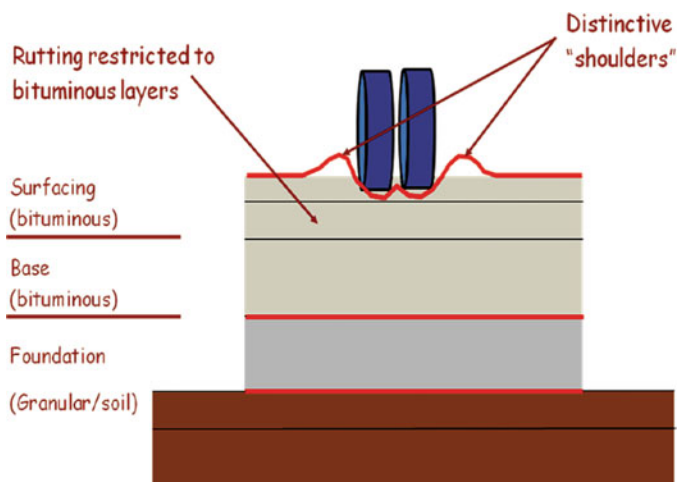


Fig. 4.3 Non-structural rutting

Permanent deformation in asphaltic mixtures due to the passage of a single vehicle is typically very small, but with a large number of vehicle passes can lead to a considerable permanent deformation.

Permanent deformation and fatigue should not be mistaken (cf. Fig. 4.4). Fatigue damage affects the rigidity of the material implying a decrease of the modulus, while permanent deformation is created by the accumulation of small irreversible strain created at each loading cycle.

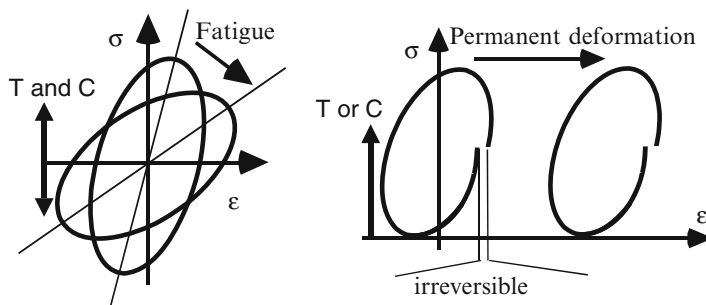


Fig. 4.4 Difference between fatigue and permanent deformation (rutting). “T” means tension and “C” compression (From Di Benedetto et al. [61])

4.1.3 Presentation of the Chapter

This chapter devoted to permanent deformation of bituminous mixtures is divided in four main sections including this introduction.

The second section gives general information on the characterisation of permanent deformation in the laboratory. Tests on both binders and mixtures are presented. Homogeneous and non-homogeneous tests are considered.

The two experimental campaigns, using Wheel tracking, tests organized and performed as part of WG3 RILEM TC 206-ATB activities are presented in Sect. 4.3. The first campaign (campaign A) is devoted to mixtures made with pure bitumen. The second (campaign B) considers mixtures with polymer modified bitumen.

The rheological modelling and numerical analysis performed by the group are developed in Sect. 4.4.

4.2 Characterisation of Permanent Deformation/Rutting in the Laboratory

4.2.1 Homogeneous and Simple Empirical Material Characterisation Tests

4.2.1.1 Tests on Binders (and Mastics)

Many different empirical tests have been used to characterise the physical properties of bituminous binders. These are carried out under particular temperatures and loading time conditions. Some of these tests could be considered as direct measurement of bitumen’s resistance against permanent deformation. Although the tests in the following section relate to those which are carried out

on binders, many of the tests can also be carried out without modification on mastics. A list of the tests covered in this section is as follows:

Penetration Test

Softening Point

Cohesive Strength

Ductility Test

Direct Tension Test

Shear Rheometry including Viscosity and Complex Viscosity Testing

Cyclic/Repeated Creep Recovery Tests

Penetration and Ring and Ball (RB) Softening Point Tests

The penetration test can be considered to be an indirect measurement of the viscosity of bitumen at 25°C. The details of the test procedure are given in BS EN 1426:2007 [30]. The penetration (“pen”) is the distance travelled by the needle into the bitumen sample under given loading conditions and is measured in decimillimetres (dmm). In this test, harder bitumens have lower values of penetration, whereas the softer bitumens exhibit higher penetration values.

The consistency of penetration grade or oxidised bitumen can be measured by determining its softening point. In this test, a steel ball is placed on a sample of bitumen which is contained within a brass ring. This set up is then suspended in a water or glycerol bath. Water is used for bitumen with softening points of 80°C and below and glycerol for softening points above 80°C. The bath temperature is then raised at 5°C per minute. The bitumen softens and eventually deforms slowly with the ball through the ring. At the moment the bitumen and the steel ball touch a base plate 25 mm below the ring, the temperature of the bath is recorded. The temperature that is recorded is designated the softening point and it represents an equiviscous temperature. The full details of the test can be found in BS EN 1427:2007 BS 2000–58:2007 [31].

The consistency of bitumen at the softening point has been measured by Pfeiffer and Van Doormaal [107] in terms of its penetration value. They used a specially prepared extra-long penetration needle and found a value of $\text{pen} = 800$ dmm for most, but not all bitumens. The exact value has been found to vary with penetration index and wax content. The viscosity of bitumen at the softening point has also been measured and found to be 1,200 Pa s (12,000 poise) for the majority of bitumens.

The softening point has also been linked directly to rutting performance in the CEN binder specifications (Dreessen et al. [73]). The higher the softening point of the binder, the better the rutting resistance of the associated asphalt mixture is expected to be.

Ductility Tests (DT)

Cohesive Strength (Ductility Test)

The cohesive strength of bitumen is characterised by its ductility at intermediate temperature. In the ductility test, three ‘dumb-bells’ of bitumen are immersed in a water bath and stretched at a constant rate of 50 mm per minute until fracture occurs. The distance the specimen is stretched before failure is reported as the ductility. The test temperature is adjusted depending on the penetration of the bitumen being tested. The test temperature is 10°C for 80–100 pen binders, 13°C for 60–70 pen and 17°C for 40–50 pen. Under these conditions, the test is able to discriminate between bitumens of different cohesive strengths [5].

Force Ductility Test (FDT)

A European version of the ductility test has been developed, which is called the force ductility test. The test procedure is very similar to the traditional method. Samples of bitumen are formed into ‘dumb-bells’ and then immersed in a water bath and stretched at a constant rate of 50 mm per minute until fracture occurs. The distance that the specimen is stretched before it breaks is reported as its ductility. The test temperature is again adjusted based on the penetration of the bitumen tested. Under these test conditions, the test has been found to discriminate between bitumens of different cohesion strengths.

However, some standard bitumens form a very thin thread at large values of strain. This can produce very misleading results as the cohesive strength is essentially zero. It has also been shown that the modification of bitumen with elastomers produces much thicker threads in the FDT, indicating a greater cohesion than found in unmodified binders.

To quantify this effect, the force required to stretch the bitumen sample is recorded during the test. The area below the resulting force-distance curve can then be calculated and represents the cohesive energy. Penetration grade bitumen and polymer modified bitumen produce significantly different force ductility curves (BS EN 13589: 2003 [29]).

Direct Tension Test (DTT)

The DTT is used to determine the tensile strength of a binder. The comprehensive method is established in AASHTO T 314–02 [1]. This test method covers the determination of the failure strain and failure stress of asphalt binders by means of a direct tension test. These parameters give an indication of a binder’s resistance to permanent deformation. The test can be used with either unaged or aged material. The test apparatus is designed for testing within the temperature range from +6 to –36°C. Figure 4.5 shows the cross-section of trimmed DTT specimen prior to testing.

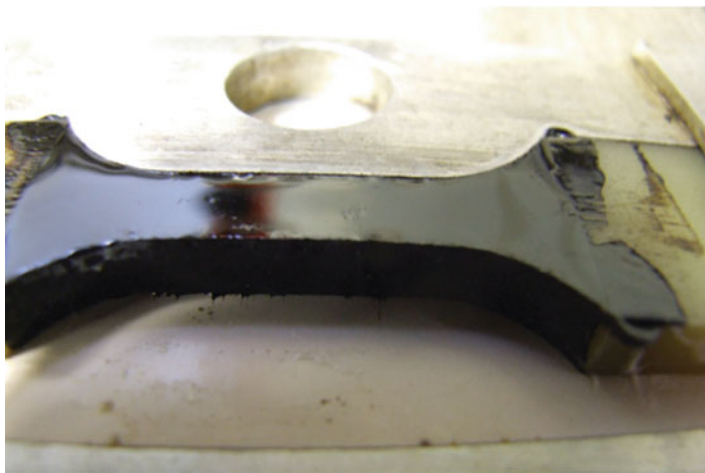


Fig. 4.5 The cross-section of trimmed DTT specimen

Viscosity and Complex Viscosity Testing

Steady Shear Viscosity

Steady shear viscosity (often called “viscosity”) is a fundamental characteristic of a bitumen as it determines how the material will behave at high temperature and/or low frequency if the behaviour is purely viscous, which is not the case for highly modified bitumens. The basic unit of viscosity is the Pascal second (Pas). The absolute or dynamic viscosity of a bitumen measured in Pascal seconds is the shear stress applied to a sample of bitumen in Pascal divided by the shear rate per second; 1 Pas = 10 P (poise). The absolute viscosity of bitumen can be measured using sliding plate, rotational or annular rheometers (among others).

Kinematic viscosity is related to dynamic viscosity by the following equation:

$$\text{Kinematic viscosity} = \frac{\text{dynamic viscosity}}{\text{density}} \quad (4.1)$$

The most commonly used rheometer is the rotational rheometer (Read and Whiteoak [113]).

Steady state flow curves can be produced by progressively increasing the shear stress applied to the sample. The steady shear state is one in which the viscosity remains constant with increasing shear (or flow rate). This method can be used to assess if a binder exhibits Newtonian flow behaviour. If so, the viscosity (equal to shear stress divided by shear strain rate) will remain constant with time. If it increases, it is said to exhibit shear thickening behaviour, and if it decreases it exhibits shear thinning behaviour (Partal et al. [105]).

Linear Viscoelasticity (LVE) and Complex Rheological Parameters

LVE materials respect the Boltzmann superposition principle (Boltzmann [17]). Bitumens are linear viscoelastic materials for “small” strain amplitude loading (Di Benedetto and Corte [62]). Their behaviour is a mixture of both viscous and elastic modes. The degree to which their behaviour is viscous or elastic is governed by both temperature and loading time. At high temperatures and/or long loading times, fluid type linear viscoelastic materials behave as viscous liquids and at low temperatures and/or short loading times they behave as elastic (brittle) solids. At intermediate loading temperatures or times they behave as visco-elastic materials. In order to characterise the LVE behaviour it is convenient to use the complex shear modulus (G^*) or the complex viscosity (η^*) or the complex Young modulus (E^*). Complex parameters are obtained when considering sinusoidal loading. Complex modulus expressed as a function of frequency and temperature is a key parameter to characterise bitumen (or mastic) behaviour. These parameters are defined in many publications [59, 62–66, 80]. Only main definitions are given Eqs. 4.2a, 4.2b, 4.2c, 4.2d and 4.2e

$$E^* = E_1 + jE_2 \quad (4.2a)$$

$$E^* = |E^*|e^{j\phi_E} \quad (4.2b)$$

$$G^* = |G^*|e^{j\phi_G} = G_1 + jG_2 \quad (4.2c)$$

$$v^* = |v^*|e^{j\phi_v} = v_1 + jv_2 \quad (4.2d)$$

$$\eta^* = G^*/(j\omega) \quad (4.2e)$$

In the case of isotropy one obtains:

$$G^* = E^*/(2(1 + v^*)) \quad (4.3)$$

Where j is the complex number ($j^2 = -1$), ω is pulsation. E^* , G^* , v^* , η^* are the complex Young's modulus, the complex shear modulus, the complex Poisson's ratio and the complex viscosity, respectively. $|X^*|$ and ϕ_X are the norm and phase angle of complex number X . X_1 and X_2 are the real and imaginary parts of X .

Cyclic Shear Rheometry is the type of test that is commonly used to obtain complex shear modulus (or complex viscosity) of fluid type materials. It involves an oscillatory sinusoidal loading applied to a bitumen sample sandwiched between two platens. The typical arrangement of a dynamic shear rheometer (DSR) is a fixed lower plate with an oscillating upper plate through which the shear force is applied to the specimen.

Zero Shear Viscosity

The Zero Shear Viscosity (ZSV) or the viscosity at zero shear rate is an intrinsic property of a bituminous binder that has been suggested could be an appropriate parameter of rutting characterisation, particularly for unmodified binders (Phillips and Robertus [109]; Sybilski [118]). This ZSV corresponds to the Newtonian viscosity obtained for steady shear tests. It can also be obtained during cyclic loading, when frequency (or pulsation) tends toward zero. Equation (4.3) can then be considered. It should be underlined that the limit only exists if the material has fluid type behaviour, which is not the case for example for polymer modified bitumens.

From Eq. (4.2e) equation we get:

$$\eta_0^* = \lim_{\omega \rightarrow 0} \eta'(\omega) = \lim_{\omega \rightarrow 0} \frac{G''(\omega)}{\omega} \quad (4.4)$$

Where η_0^* is the ZSV value that is a real number only if the behaviour is of “fluid type” at high temperature and/or low frequency.

Practical Use of Viscosity and Complex Viscosity Parameter

(a) Zero shear viscosity

Some authors consider that conventional pavement design is normally based on the understanding that wheel loading occurs in the linear region, then only the viscosity that occurs in the linear region, which is the ZSV, needs to be considered to evaluate the permanent deformation resistance of asphalt mixtures. In the linear region, the ZSV is independent of shear rate and it reflects dissipated motions in a negligibly perturbed, equilibrium ‘no flow’ structure (Phillips and Robertus [109]). In the case of modified binders, a Low Shear Viscosity should be used instead (BitVal [13]).

Reliable measurement protocol of the ZSV has been developed by Desmazes et al. [55]. This involves performing creep flow followed by a creep recovery test on conventional, multigrade and styrene-butadiene-styrene (SBS) polymer modified bitumens using a DSR. In creep testing, the applied shear stress has a constant value τ_0 , and the resulting shear strain γ is measured as a function of time. The creep compliance $J(t)$ is defined as the ratio of the shear strain and the applied shear stress.

Difficulties encountered in fully understanding the rheological behaviour of bitumen has led to some researchers to look at critical points in the Superpave system. One approach has led to the definition of the creep resistance of the binder, interpreted in terms of viscosity and more specifically the Zero Shear Viscosity (ZSV). This ZSV is identified from the flow which is established from a state of near equilibrium, in which the internal structure of the material remains unchanged (Giuliani and Merusi [83]). It is possible to evaluate only the viscous behaviour of the binder, which according to linear viscoelasticity

theory, is directly equal to the ZSV (Eq. 4.4). Therefore from the steady shear condition the ZSV can be determined (BitVal [13]). Results from ZSV are also proposed by Anderson et al. [6] and De Visscher et al. [50].

(b) $IG^*/\sin\delta$ parameter

The Superpave specification parameter $IG^*/\sin\delta$ was identified as the term to be used for high temperature performance grading of paving binders and rating the binders for their rutting resistance. The ineffectiveness of $IG^*/\sin\delta$ in capturing the high temperature performance of paving binders for rating their rutting resistance became a matter of significant concern as more and more polymer-modified binders were tested for their performance. The failure of this parameter was demonstrated in several studies (Delgadillo et al. [52]), including through the laboratory testing during the National Cooperative Highway Research Program (NCHRP) Project 9–10 (Bahia et al. [10]). They performed repeated creep and recovery test for binders (RCT) and it was suggested as a possible means to estimate the rate of accumulation of permanent strain in the binders. Though the RCT (see next section) test gives realistic information, the procedure is time-consuming.

Cyclic Repeated Creep Recovery Tests

Many researchers have conducted various different types of creep testing using the DSR. These have consisted of creep with or without recovery and repeated creep tests to investigate steady state shear deformation behaviour of bituminous binders.

Phillips and Robertus [109] investigated the correlation of asphalt mixture rutting with binder zero viscosity. They performed small amplitude, low frequency oscillatory tests and low stress creep with recovery tests, with an applied stress of 30–199 Pa at 40°C on conventional unmodified bitumen, multigrade bitumens and SBS polymer modified bitumens containing various polymer contents. This test only perturbed a bituminous material slightly and enabled the ZSV to be measured. They found that when mixtures made with these binders were tested for rutting using a Laboratory test track (LTT) carousel, the rutting rate correlated well with the ZSV of the binder at the same temperature. Phillips and Robertus [108] also carried out dynamic creep testing on the same asphalt mixtures at a temperature corresponding to an iso-viscosity condition, $ZSV = 10^6$ Pa.s. Again they found that the creep rate correlated well with the ZSV of both unmodified and polymer modified binders.

Desmazes et al. [55] carried out creep flow and then creep recovery tests at 40°C using a controlled stress rheometer on conventional, multigrade and SBS polymer modified binders in order to develop a test method. They used a stress level of 20–50 Pa for modified binders and up to 5,000 Pa for unmodified binders. The creep and creep recovery times were long enough to allow steady state conditions and as long as this was reached, it was possible to measure the ZSV.

Creep testing using the DSR has also been carried out by Carswell and Green [33], using stress levels of 100 and 15,000 Pa at temperatures of 45 and 60°C.

Collop et al. [42] conducted creep tests on both 50 and 100 pen bitumens at 20°C using a DSR over a wide range of stress and strain levels to investigate the steady-state deformation behaviour in the linear and non-linear regions. It was found that the steady-state deformation behaviour of the two penetration grades was similar. They also found that at stress levels less than approximately 50 kPa, the deformation behaviour was linear viscous flow. At stress levels greater than 100 kPa the deformation behaviour was found to be non-linear creep behaviour.

Many researchers have used the DSR for performing cyclic creep recovery tests. Examples of such work include Carswell and Moglia [34], who used a repeated creep (creep pulse) test using a DSR to determine an accumulated strain parameter. This test involved applying a load of 300 Pa for 1 s and then allowing recovery for 9 s. This was carried out for 100 cycles.

Vlachovicova et al. [125] performed creep and recovery tests at 60°C on base bitumen (200/300 penetration grade) modified with different amount of radial styrene-butadiene-styrene (SBS) copolymer. They investigated various time durations of creep and recovery. The impact of waiting time between tests on creep compliance and ZSV and the influence of stress levels ranging from 25 Pa up to 5 kPa were investigated. Their work found that the estimated ZSV and the steady-state compliance were stress dependent above a stress level of 25 Pa. This seemed to be due in part to the fact that steady-state flow was not reached, especially for short cycles (1 s loading time and 9 s unloading time). They recommended that further researchers consider creep and recovery cycles with higher stress levels.

In the US, work was carried out to produce a new high-temperature binder specification using multi-stress creep and recovery. The objective of this work was to produce a PG binder test that was both performance-based and blind to modification type. The Multi Stress Creep and Recovery (MSCR) test was developed as a replacement for the existing AASHTO M-320 high-temperature binder test. In the work carried out by D'Angelo [47], both neat and polymer modified binders were evaluated in the development of a new binder test to determine the high -temperature rutting of property for binders. The DSR was used as the equipment for these tests. The test involved a 1 s creep loading with a 9 s recovery period. This creep loading was carried out over a range of stress levels; 0.025, 0.050, 0.1, 0.2, 0.4, 0.8, 1.6, 3.2, 6.4, 12.8 and 25.6 kPa at ten cycles per stress level. The test is started at the lowest stress level and increased to the next stress level at the end of every ten cycles with no time lag between cycles. The average non-recovered strain for the ten creep and recovery cycles is then divided by the applied stress for those cycles yielding the nonrecoverable compliance, J_{nr} . The determination of J_{nr} is shown in Fig. 4.6.

$$J_{nr} = \text{average } \gamma_u / \tau,$$

γ_u = unrecovered strain from the end of the 9 s recovery portion of the creep and recovery test

τ = shear stress applied during the 1 s portion of the creep and recovery test.

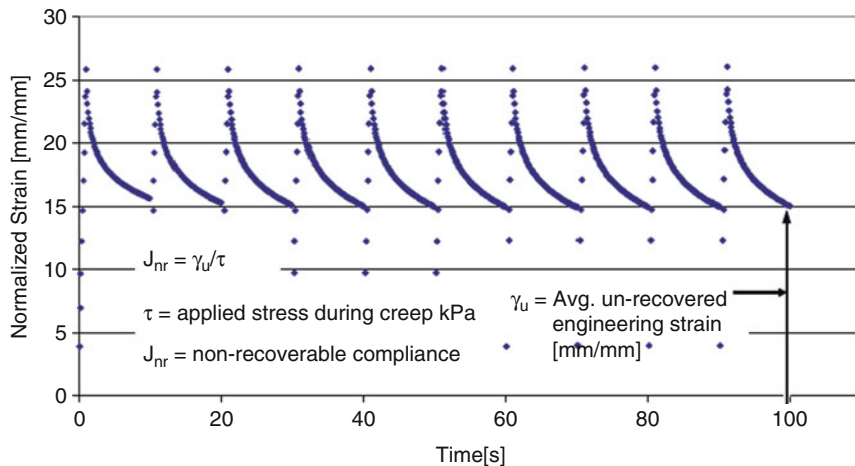


Fig. 4.6 Plot showing the determination of J_{nr} and the recovered strain from the MSCR test

Various evaluations have shown that the MSCR test is able to distinguish the difference in rutting potential between various binders, both modified and unmodified (D'Angelo et al. [45–48]). The evaluations were carried out using several different testing types, including loaded wheel testers, accelerated load testers and actual pavement sections. Stress dependency of the binders was identified which established the criteria to evaluate the different mix tests and their relationship to binder properties. The overall findings showed that the nonrecoverable compliance J_{nr} would be a good replacement for the SHRP high-temperature binder criteria.

4.2.1.2 Tests on Mixtures

Permanent deformation occurs in asphalt mixtures as a result of densification and shear stress. The majority of densification of asphalt from traffic loading occurs in the first year of construction (Newcomb et al. [100]). Permanent shear deformation then occurs due to the viscous behaviour of the material.

An objective of pavement material testing is to investigate their behaviour under conditions as close to in situ as possible. However, reproducing in situ conditions in the laboratory is not easy. It also requires very complex equipment. This has meant that simplified tests have been used to investigate the permanent deformation behaviour of asphaltic mixtures. The most common tests are listed below:

- The Marshall Test
- Indirect Tensile Testing
- Indirect Tensile Stiffness Modulus
- Constant Strain Rate Tests
- Creep Testing in Pure Tension or Compression
- Indentation Testing

- Creep Recovery and Repeated Creep Recovery
- Repeated Load Axial Test
- 1D Cyclic Tension or Compression Tests with and without Recovery
- Monotonic and Cyclic Triaxial Testing
- Complex Modulus E^*
- Rotation of Axis Testing including: Repeated Simple Shear Test, Repeated Simple Shear Test at Constant Height (RSST-CH), Shear Frequency Sweep (FS-S), Hollow Cylinder Test

The Marshall Test

The concept of the Marshall test was developed by Bruce Marshall, who was a Bituminous Engineer with the Mississippi State Highway Department. In 1948, the procedure was improved by the US Corps of Engineers and developed into a mixture design criteria. Since then it has been adopted by many organisations and government departments in many countries.

The Marshall test involves the manufacture of cylindrical specimens (102 mm in diameter $\times$ 64 mm high) using a standard compaction hammer and a cylindrical mould. The specimens are then tested for their resistance to deformation at 60°C at a constant rate of 50 mm/min. The jaws of the loading rig confine the majority of the circumference of the specimen, the top and the bottom of the specimen being unconfined. This means that the stress distribution in the specimen is extremely complex. Two properties are then determined: the maximum load carried by the specimen before failure ('Marshall stability') and the amount of deformation of the specimen before failure occurred ('Marshall flow'). The ratio between the two is known as the 'Marshall quotient'.

Although the Marshall test is widely used, it is important to understand its limitations. Research carried out at the University of Nottingham (Brown et al. [23]) compared the mechanical properties of various mixtures, using repeated load triaxial tests, triaxial creep tests, uniaxial unconfined creep tests and Marshall tests. It was found that the Marshall test is a poor measure of resistance to permanent deformation and does not rank the mixtures in order of their resistance to deformation. The repeated load triaxial test was found to give more realistic results.

Indirect Tensile Testing

Indirect Tensile Stiffness Modulus (ITSM)

Although this test does not directly deal with permanent deformation, it has always led on as it gives a main characteristic of asphalt, which is considered as closely linked. This method is employed to measure the stiffness modulus of an asphalt mixture. It uses cylindrical samples laid on the side as shown in Fig. 4.7.

Fig. 4.7 Indirect Tensile Stiffness Modulus (*ITSM*) equipment



During this test, the stress distribution is assumed to be known in the centre of the specimen under the hypothesis of isotropic linear elastic behaviour. Transducers are used to measure the horizontal deformation, enabling to deduce the horizontal strain, as the diameter of the specimen is known. Thanks to Poisson's law, horizontal stress can be determined. By knowing the cross-sectional area, the measurement of the vertical force applied gives the vertical stress in the centre. As the vertical strain and horizontal strain are now known, we can calculate the stiffness modulus following Hooke's law. As this method is non-destructive, it does not require additional samples (BS DD 213:1993 [26]; BS EN 12697-26:2004 [28]).

Uniaxial Constant Strain Rate Tests

Static uniaxial tests are the simplest tests used for characterisation of the behaviour of materials like asphaltic mixtures. These tests are either conducted under constant stress (creep test) or constant strain rate conditions, in either tension or compression. Compressive tests are the more common of the two. In these tests, cylindrical specimens are generally used and it is advised to use a friction reduction system

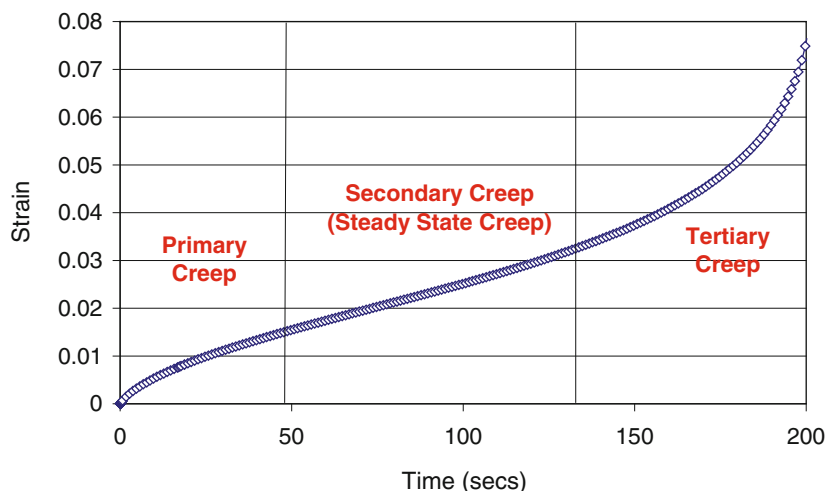


Fig. 4.8 Typical creep curve for asphaltic mixture

between the loading platens and the specimen to prevent the sample from barrelling due to non-uniform stress distributions.

Monotonic constant strain rate tests can be used in conjunction with constant stress creep tests to study the steady-state deformation behaviour of bitumen, idealised and realistic asphalt mixtures (Deshpande and Cebon [53]; Collop and Khanzada [41]). Cheung [38] and Deshpande [54] found that the constant strain rate test and the constant stress creep test are complimentary experimental tools for the investigation of the steady-state deformation behaviour of bitumen and idealised asphalt mixtures.

Creep Testing in Pure Tension or Compression

In a static uniaxial creep test, a vertical stress is applied rapidly to the specimen and then held constant. The most basic of simple compressive creep tests is outlined in BS 598–111:1995 [25]. The axial and radial deformations are measured, along with the axial load as a function of time. The deformations can then be converted to strains. Figure 4.8 shows a schematic of a typical creep curve for a bituminous mixture. The first part of the creep curve belongs to the instantaneous strain, composed of elastic and plastic strain, which is also known as the time-independent strain since it occurs during application of load. This is then followed by the time-dependent strain, which is composed of delayed elastic and viscous strains. In total, the creep curve can be divided into three sections. The primary region where the strain rate decreases with time, the secondary region where the strain rate remains constant and the tertiary region, where the strain rate increases with time, heading towards failure.

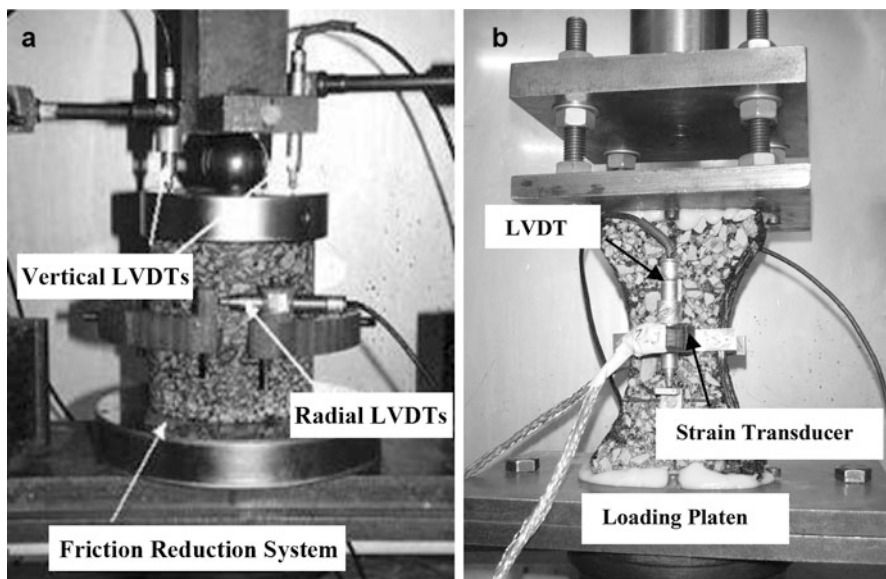


Fig. 4.9 Uniaxial test set-up (a) compression, (b) tension

Examples of uniaxial test set-ups for compression and tension are shown in Fig. 4.9. An early development of the creep test was the static creep test developed by the Shell organisation. This test was used to try to judge the stability of mixtures in terms of their resistance to permanent deformation and allow the prediction of rut depth in pavements (Hills [88]). Experimental results were presented in plots of stiffness modulus of the mixture S_{mix} as a function of stiffness modulus of the bitumen S_{bit} . The stiffness of the mixture was defined as a constant stress divided by the nominal strain in the mixture. The stiffness of the bitumen was obtained from the Van der Poel nomograph, using loading time, temperature and the penetration and softening point of bitumen. For short loading times and or low temperatures, it was found that the stiffness modulus of a mixture was a function of the stiffness modulus of the bitumen and volume fraction of aggregate. However, at higher temperatures and or longer loading times, in addition to the effect of the aggregate volume concentration, the gradation, shape, surface texture and compaction level were found to be important in determining the stiffness of the mixture.

There are some differences between the static uniaxial creep test and realistic conditions. In pavements, the materials are confined and loaded dynamically, whilst in the creep test the material is unconfined and the load is static. For the same accumulated loading time, the permanent deformation of asphaltic mixtures under dynamic loading has been found to be greater than under static loading (Bolk and Van De Loo [16]; Brown et al. [22]; Monismith and Tayebali [97]; Gibb [82]). This difference has also been found to increase with increasing number of load passes (Brown et al. [19]; Gibb [82]). This has been attributed to the contact between aggregate particles, which causes the plastic deformation during each load pass.

The differences between behaviour under static and cyclic loading also vary with mixture type (Van de Loo [124]) and confining stress level (Monismith et al. [97]), with differences decreasing as confining stress increases.

Indentation Testing

Indentation tests provide a cheap and easy method to measure the mechanical properties of materials and also serve to validate multi-axial constitutive models of materials (Ossa [104]). The standard indentation test on creeping solids involves either applying a constant load and measuring the indentation with time or pressing the indenter in at a prescribed rate, which increases the load as a function of time. Many researchers have developed models to relate the indentation pressure to constitutive models of materials to help interpret these results. Tabor [119], proposed empirical relations to correlate the indentation pressure for rate independent strain hardening materials to the uniaxial tensile response. Mulhearn and Tabor [98] extended these empirical relationships to power-law creeping materials. Full details of the theories concerning indentation behaviour of creeping solids can be found in the work carried out by Ossa [104].

Creep Recovery and Repeated Creep Recovery

Single and/or cyclic creep recovery tests have been used by researchers for the characterisation of elastic, plastic, delayed elastic and viscous strain components in asphaltic mixtures (Sides et al. [115]; Drescher et al. [74]; Lu [95]). The strain components can be studied as a function of stress level, temperature, loading time, unloading time and the number of cycles. In a single creep recovery test, a stress is applied rapidly to a specimen and then held constant. The specimen is allowed to creep for a certain length of time and then the stress is rapidly released to allow the elastic and delayed elastic components of the deformation to recover. In cyclic creep recovery tests, the loading and unloading is repeated for a certain number of cycles which allows the study of the deformation components as a function of a number of cycles.

Uniaxial or triaxial single creep recovery tests were performed over a range of deviatoric stresses, temperatures, stress ratios and total strains before unloading by Ossa [104] to investigate the recovery behaviour of low volume fraction and fully dense idealised mixtures. It was observed that in a similar way to bitumen, for both uniaxial and triaxial conditions, the recovered strain increased linearly with increasing total strain before unloading. The slope of the line, named the recovery constant, was found to be independent of temperature, deviator stress level and stress ratio. However, the recovery constant was found to decrease with increasing volume fraction of aggregate. A model was developed to capture the recovery strain versus time history of the mixtures. By measuring the radial strain of the specimen and calculation of the shear and volumetric strains, it was found that the volumetric

strain varied linearly with the shear strain with the same slope during creep and recovery.

Repeated Load Axial Test

This test is convenient since the test arrangement is simple. The pulsed load is considered to simulate the real-life condition of traffic loading. A cylindrical test specimen is placed between two parallel loading platens at a target temperature. Repeated load is applied, and then vertical axial deformation is measured and plotted against load cycles. However, this test has been criticised as a considerable degree of confinement pressure is not taken into consideration (BS EN12697-25:2005 [32]).

Cyclic tests have also been used to study the deformation behaviour of pavement materials under more realistic loading conditions (Brown [21]). In the cyclic uniaxial tests, the axial load is applied repeatedly with or without a rest period between the pulses. The maximum deformation from each load pulse is recorded and plotted against the number of cumulative cycles. The creep curve for the cyclic creep test is similar to that of the static creep test (see Fig. 4.8).

Cyclic Tension or Compression Tests with and Without Recovery

The cyclic “permanent deformation” test developed at University of Lyon/ENTPE (Dongmo [70]; Nguyen [101]; Nguyen et al. [102]) consists of applying haversine compression (or tension) loading stress periods and rest periods ($\sigma_I(t) = 0$) to the specimen. The experimental device is presented in Fig. 4.10, and the loading protocol is shown in Fig. 4.11. Axial and radial strains (ε_I and ε_2) as well as axial stress ($\sigma_I(t)$) are measured. Different frequencies and temperatures are considered. The evolutions with time of the three measured values, during cyclic loading, are given in the following equations:

$$\sigma_I(t) = \sigma_{0I}(1 + \sin(\omega t + \phi_E)) \quad (4.5)$$

$$\varepsilon_I(t) = \varepsilon_I(N, \tau) = \alpha_I(N)\tau + \varepsilon_I^P(N) + \varepsilon_{0I}(N) \sin(\omega\tau) \quad (4.6)$$

$$\varepsilon_2(t) = \varepsilon_2(N, \tau) = \alpha_2(N)\tau + \varepsilon_2^P(N) - \varepsilon_{02}(N) \sin(\omega\tau + \phi_v) \quad (4.7)$$

where τ , $\alpha_I(N)$, $\varepsilon_I^P(N)$, $\varepsilon_{0I}(N)$, σ_{0I} , ϕ_E are presented in Fig. 4.11.

A noticeable characteristic of this test is that it allows measurement of both the behaviour in the small strain (linear) domain (using the 3rd from the right terms of Eqs. (4.6) and (4.7)) and the behaviour in the non-linear domain (using the 1st and 2nd from the right terms of Eqs. (4.6) and (4.7)). The axial and radial permanent deformations, $\varepsilon_I^P(N)$ and $\varepsilon_2^P(N)$, which can reach some 10^{-2} m/m and the sinusoidal

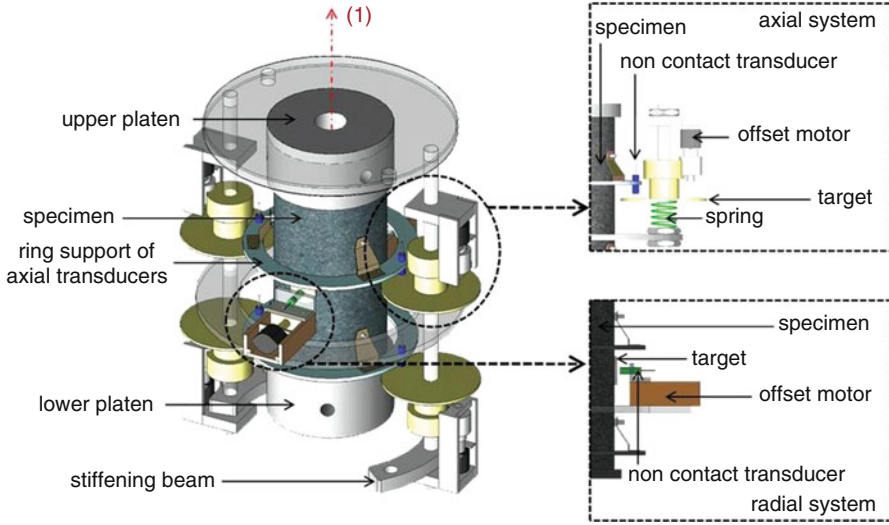


Fig. 4.10 Experimental device used at University of Lyon/ENTPE for both tension-compression (push-pull) test, in the linear domain, and cyclic “permanent deformation” tests, in the non-linear domain

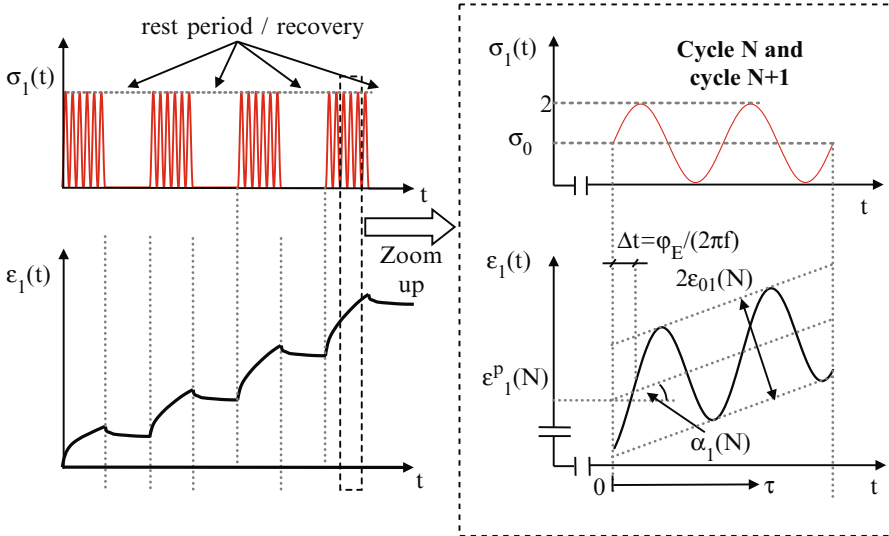


Fig. 4.11 Loading protocol and zoom in on the cyclic period for cyclic permanent deformation tests

amplitude strain components $\varepsilon_{01}(N)$ and $\varepsilon_{02}(N)$, (around some 10^{-5} m/m) are both measured (Dongmo [70]; Nguyen [101]). Sinusoidal components of the axial stress and of the axial and radial strains give the complex Young’s modulus (E^*) and the

complex Poisson's ratio (ν^*). During rest periods (recovery) (Fig. 4.11), all the sinusoidal components are nil ($\sigma_{01} = \varepsilon_{01} = \varepsilon_{02} = 0$).

Performing such tests gives valuable information on the evolution of permanent deformation on the considered range of frequencies and temperatures. It allows improvement of rheological models such as the DBN model developed at University of Lyon/ENTPE (Neifar and Di Benedetto [99]; Di Benedetto et al. [63, 64]).

A very noticeable result observed from different experimental campaigns is that the Time-Temperature Superposition Principle (TTSP) is verified in the non-linear domain. This property is validated because different cyclic permanent deformation tests (Fig. 4.11) having the same loading histories in the equivalent time ($t_{eq} = t/a_T$, where a_T is the shift factor) give very close results. This TTSP property can be observed in Fig. 4.12, where a couple of tension tests on a French BBSG mixture (Béton Bitumineux Semi-Grenu), having the same “equivalent” history, are presented.

The two tests (M1C16 and M1C14) are performed at different temperatures (25°C and 16.6°C), chosen such as $a_{T1=16.6^\circ\text{C}}/a_{T2=25^\circ\text{C}} = 10$. The test at 25°C is then ten times faster than the test at 16.6°C. During the cyclic loading sequences, the frequency of haversine stress is 10 Hz for the test at 25°C and 1 Hz for the test at 16.6°C. The values of the shift factor a_T are obtained from the test in the linear domain. It comes:

$$\sigma_{1 \text{ test at } T1}(t/a_{T1}) = \sigma_{1 \text{ test at } T2}(t/a_{T2}) = \sigma_1(t_{eq}) \quad (4.8)$$

From the results presented in Fig. 4.12, the evolutions of the axial and radial strains in the equivalent time scale are very close for the two tests. These results validate the TTSP for this type of loading in the non-linear range (up to 5%). In addition a three dimensional validation is provided as axial and radial strains are involved.

Monotonic and Cyclic Triaxial Testing

In the cyclic triaxial test (Fig. 4.13), the vertical load is applied repeatedly along with a static or cyclic confining stress. Depending on the equipment, the vertical and lateral loads can be applied in various shapes such as sinusoidal, square, triangular and trapezoidal. Sinusoidal loading, although requiring more complex equipment, is best for simulating in situ loading conditions (Brown [20]). The loading time is chosen to represent that of the material under passing traffic in the pavement. Barksdale [69] has presented data with which the duration of the pulse in a repeated load test, corresponding to vehicle speed and depth in the pavement, can be determined. Various loading systems, from simple pneumatic or mechanical systems to more sophisticated servo-controlled electro-hydraulic systems, have been used to apply repeated loading, (Brown [20]; Di Benedetto and Yan [57]). ESSO Company (Azibert et al. [9]; Aussedat [8]; Célaré [35]), developed a cyclic

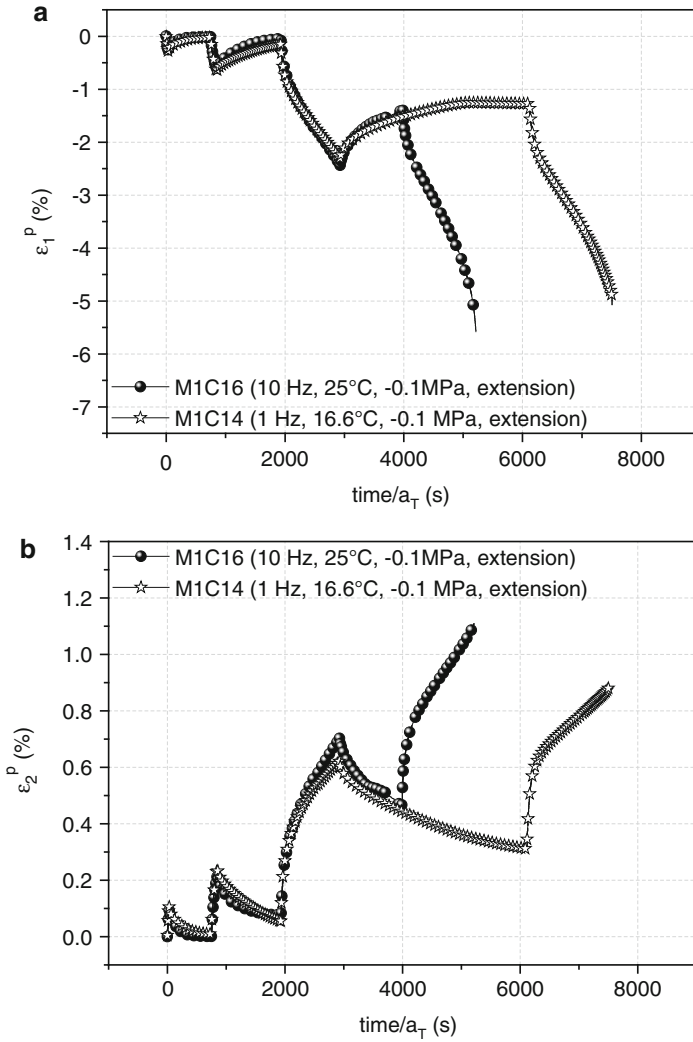


Fig. 4.12 Experimental result for one pair of “equivalent” cyclic tension tests (M1C14 test: $T = 16.6^\circ\text{C}$, $f = 1\text{ Hz}$, $\sigma_{01} = -0.1\text{ MPa}$ and M1C16 test: $T = 25^\circ\text{C}$, $f = 10\text{ Hz}$, $\sigma_{01} = -0.1\text{ MPa}$), axial (a) and radial (b) permanent deformation versus equivalent time ($t_{eq} = \text{time}/a_T$)

triaxial test allowing confinement, called “dynamic creep test”. This name is not judicious as no dynamic nor creep is applied during the tests.

In the US, the flow time (F_t) is used as a measure to distinguish the different deformation behaviour of asphalt mixtures. This measure is obtained using the triaxial static creep test (Dongre et al. [71]).

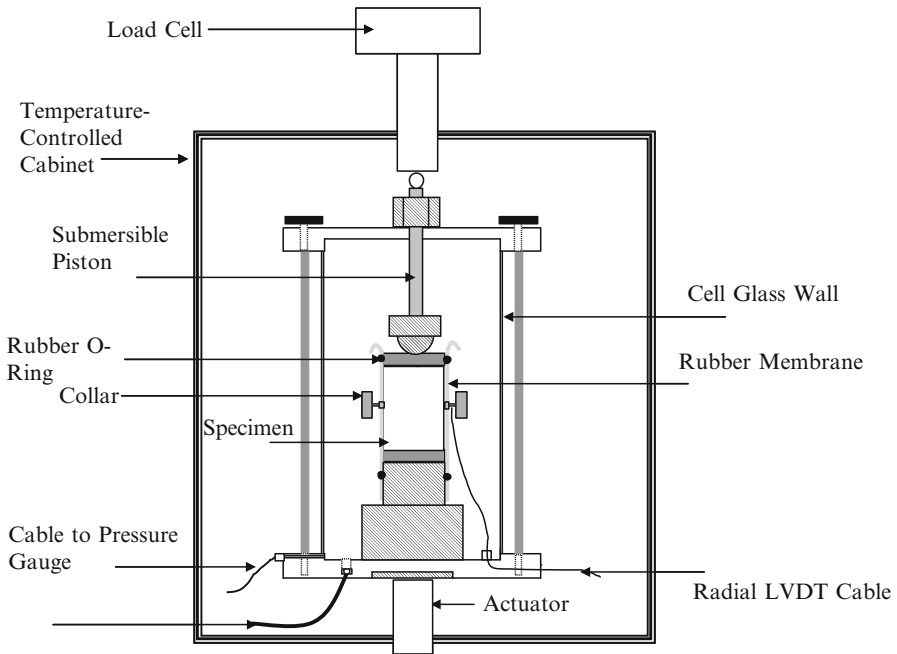


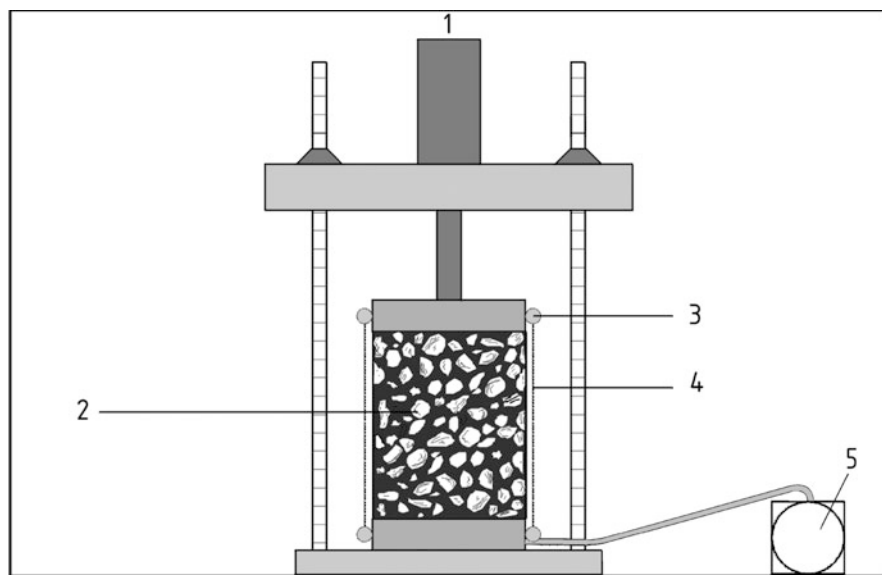
Fig. 4.13 Schematic of the triaxial cell

Vacuum Repeated Load Axial Test (VRLAT)

The aim of this device is the same as the previous one: to determine the resistance of asphalt to permanent deformation. Contrary to Tri-axial test, the confining pressure is obtained with reducing the pressure inside the specimen. For this purpose, the sample is sealed with a latex membrane and the air inside the specimen is evacuated through a porous lower platen thanks to a vacuum pump. As in the previous method, the vertical strain is monitored via a transducer. Figure 4.14 shows a schematic of this equipment and the actual test can be seen in Fig. 4.15. The suction is applied from the bottom whilst the stress is applied from the top. As this device can be mounted on a machine used for other tests, it saves money. Although a few studies have shown that the VRLAT gives good results with high air void content mixtures, it is not known whether this method can be used with more dense materials (BS EN12697-25: 2005 [23]).

Triaxial Repeated Load Permanent Deformation Test (RLPD)

Another measure of asphalt mixtures' resistance to permanent deformation is given by the flow number (F_n). This number is defined using the triaxial Repeated Load Permanent Deformation (RLPD) test. During this test, a specimen at a specific test temperature is submitted to a repeated haversine axial compressive load pulse of 0.1 s every 1 s. A specimen typically experiences three stages of flow. Firstly, there



1 Actuator for dynamic pressure; 2 Specimen put under partial vacuum; 3 'O' ring; 4 Vacuum membrane; 5 Vacuum pump

Fig. 4.14 Schematic of the Vacuum Repeated Load Axial Test (VRLAT). 1. Actuator for dynamic pressure; 2. Specimen put under partial vacuum; 3. 'O' ring; 4. Vacuum membrane; 5. Vacuum pump

is the primary region where densification occurs, then the secondary region of steady-state flow. Finally, the specimen moves into the tertiary region where shear deformation occurs. The onset of the shear deformation is related to the resistance of Hot Mix Asphalt (HMA) mixtures to permanent deformation and is designated as F_n . The resulting permanent axial strains are measured as a function of load cycles and numerically differentiated to calculate the F_n (Dongre et al. [71]).

The F_n test is used in mix design performance analysis due to its potential to rank the relative permanent deformation of HMA mixes, determine optimal binder content and also examine the effects of air void contents. Biligiri et al. [12] evaluated several different mathematical models for calculating the onset of tertiary flow. They identified a new comprehensive mathematical model, called the Francken model, to accurately determine the flow number F_n . Further details can be found in Biligiri et al. [12] and Dongre et al. [71].

Complex Modulus E^*

Complex modulus (E^*) is a commonly measured and extensively studied property of bituminous mixtures, which gives linear viscoelastic properties. The provided information and properties are suited to the mechanistic analysis and design of

Fig. 4.15 Picture of the Vacuum Repeated Load Axial Test (VRLAT)



pavements. Complex modulus is measured through the application of a sinusoidal (haversine) loading at various frequencies ω and temperatures T . The resultant viscoelastic strain response lags behind the stress in time, usually expressed in terms of phase angle ϕ . Different types of test are proposed for mixtures (Di Benedetto et al. [59]; Di Benedetto and Corte [62]). The tests can be either homogeneous, which gives direct access to the rheological behaviour, or non-homogeneous (Fig. 4.16). Among these tests, cyclic Tension/Compression test is commonly used for mixtures (Witczak et al. [128]; Witczak and Root [129]; Di Benedetto et al. [68]).

Previous research performed by RILEM groups, using a wide range of sample shapes, testing geometries and loading conditions, showed that the complex modulus results from the bending tests, indirect tension tests and Tension/compression tests were reasonably similar within the linear small strain domain (Di Benedetto et al. [59]). It should be underlined that this result is not true for fatigue tests (Di Benedetto et al. [60]).

It is also possible to obtain information on complex modulus using wave propagation tests. This kinds of tests that are “dynamic”, need back analysis to be interpreted (Stephenson and Manke [117]; Whitmoyer and Kim [126]; Lacroix et al. [92]; Hochuli et al. [89]; Di Benedetto et al. [67])

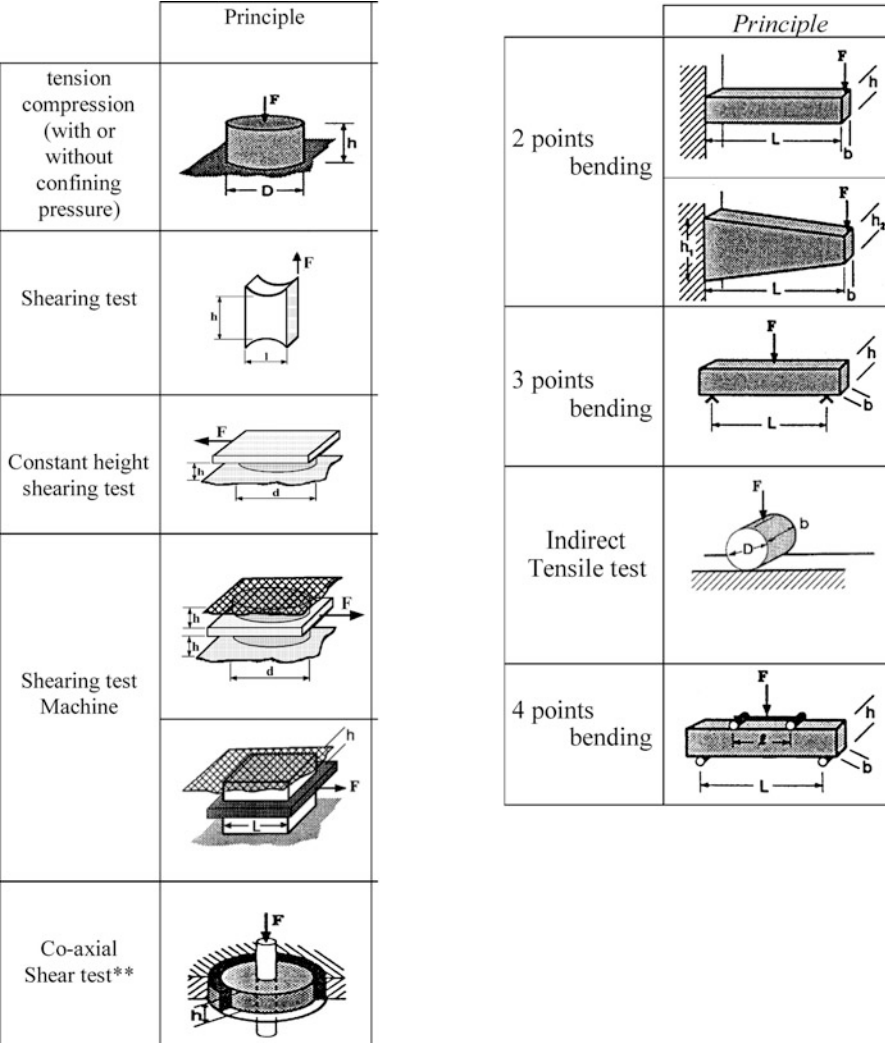


Fig. 4.16 Different tests used for complex modulus measurement: homogeneous test (*left*) and, non-homogeneous tests (*right*)

Many attempts are proposed in literature to correlate complex modulus with permanent deformation ability of bituminous mixtures. When considering Fig. 4.4, it could be thought that the irreversible part of strain (creating permanent deformation) may not be linked with viscoelastic properties characterized by the shape of the elliptical loop. This last consideration was confirmed experimentally by Nguyen [101] who obtained very different irreversible parts for cyclic loadings having same reversible strain amplitude. The identical reversible strain amplitude is obtained by changing both frequency and temperature.

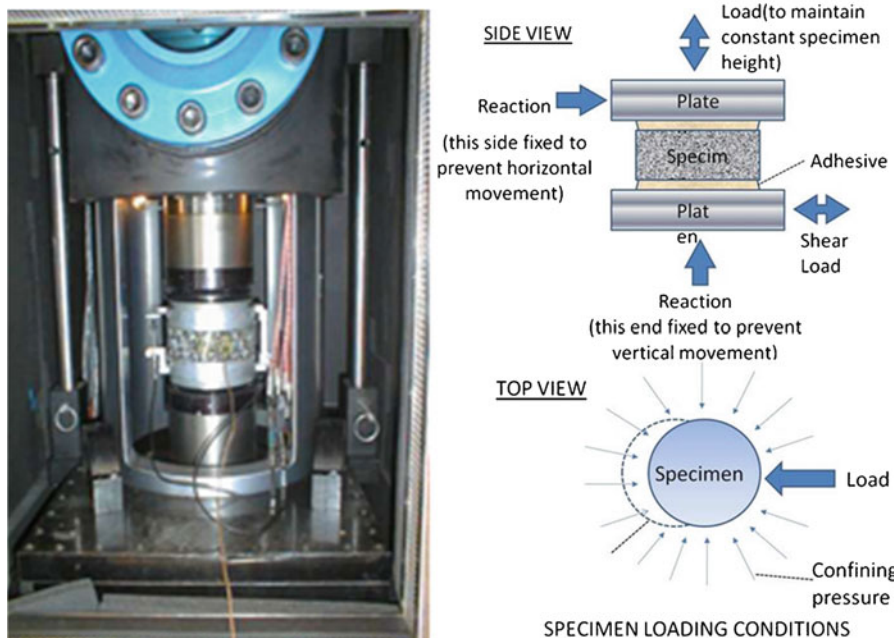


Fig. 4.17 The Simple Shear Test

Rotation of Axis Testing

A moving vehicle creates in the pavement layers stress and strain fields whose principal axes rotate in a cyclic way (Di Benedetto and Corte [62]). In order to simulate correctly this rotation of axes effect on rutting, tests introducing this rotation have been developed.

Repeated Simple Shear Test (SST)

As part of the Strategic Highway Research Program (SHRP) the SST was developed (Fig. 4.17). It is used to measure the permanent deformation properties of asphalt mixtures. This test measures the primary mechanism contributing to rutting, which is shear distortion. The testing system consists of two orthogonal tables connected to two perpendicular load actuators, situated in an environmental chamber, which can provide accurate temperature and air pressure control. The specimen is held on the shear table, which is actuated to impart a shear load. Vertical and shear deformations are measured. The Simple Shear Test (SST) is capable of applying various modes of loading. The following sections provide a brief description of two of the tests that can be performed using this piece of equipment. More detailed descriptions can be found in Harvey et al. [86].

Repeated Simple Shear Test at Constant Height (RSST-CH)

The RSST-CH is used to predict the rutting performance of asphaltic mixtures. Cylindrical specimens which can be cored from the field or laboratory compacted can be used. The diameter of the specimen can be 150 or 200 mm and their height should be less than 1/3 of the diameter. These figures also depend on maximum aggregate size (Harvey et al. [84]). The test is conducted at specified temperatures. The specimen is subjected to repeated haversine shear stress followed by a rest period. The magnitude of the shear stress, the loading duration and the rest period can all be varied. The typical loading conditions that are used for highway applications in US are a shear stress of 68.9 kPa, a load duration of 0.1 s and a rest period of 0.6 s (Harvey et al. [85]). Deformations during the test are measured. The specimen dilates under shear stress, which leads to an increase in the specimen height. As a result the variable axial stress is feedback controlled to maintain a constant specimen height. The resilient and permanent deformations are then calculated. Typically, the test is run for 5,000–10,000 repetitions. The permanent shear strain versus load repetition is the plotted, which has three regions, similar to static and cyclic creep tests (see Fig. 4.8). The resilient shear stiffness for the given load condition and test temperature can also be calculated. The results gained from the RSST-CH can then be used to rank mixtures for expected performance, or to estimate their performance.

Shear Frequency Sweep (FS-S)

The FS-S test is the same as the RSST-CH test in terms of specimen dimensions, constant height condition, instrumentation and testing equipment. The differences occur in the loading and the temperatures. This type of testing is performed over a series of frequencies and temperatures. The loading is a shear stress adjusted to provide a sinusoidal shear strain wave and a variable axial load to maintain a constant specimen height. The test gives data in the form of shear stress amplitude, shear strain amplitude and the shear phase angle at each frequency. These can then be used to determine the linear viscoelastic properties of the materials (i.e. storage modulus, loss modulus, complex modulus and phase angle). A master curve of the shear stiffness can then be developed using the results of the test at different temperatures.

Hollow Cylinder Apparatus (HCA)

Over the years long hollow cylindrical specimens have been used to simulate different stress conditions on various materials. They have mainly been used to investigate a variety of stress paths in geotechnical investigations (Hight et al. [87]; Dutine et al. [75]; Duttine [76]; Blanc [14], among others), which has been done with great success.

It has also been shown that the HCA appears to have the potential to closely simulate the complex in-situ pavement stress regime induced by moving wheel loads. Previous work at the University of Nottingham and the University of

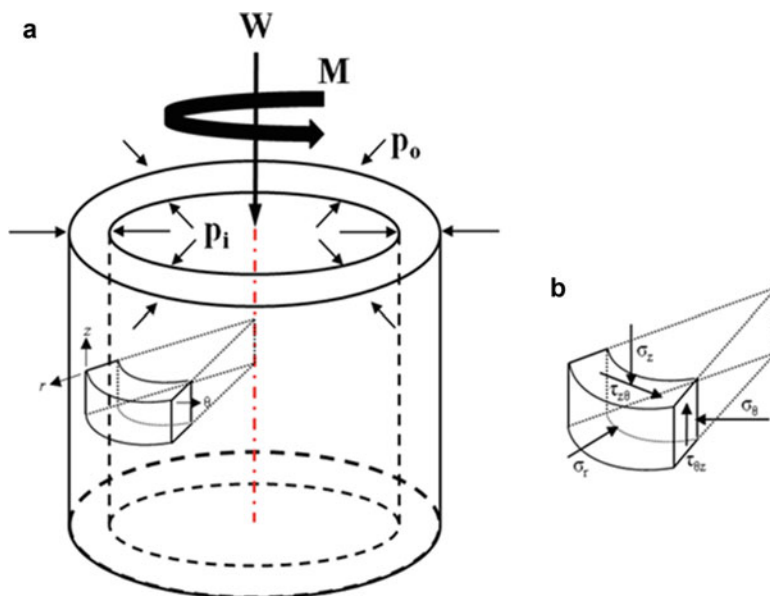


Fig. 4.18 Stress regime on an element of the wall of the HCA

California at Berkeley has shown that the equipment has the capability to generate complimentary shear stresses to the tested specimen, which allows the study of material anisotropy and the principle stress rotation effects (Richardson [114]; Alavi [4]; Chan [37]; Thom [121]; Sousa [116]).

In the HCA, the hollow cylindrical specimen can be subjected to axial (W) and torsional (M) loads, applied to the ends, whilst also being subjected to a lateral confining pressure (P_o , P_i). This configuration allows independent control of the magnitude of the three principal stresses and the rotation of the major-minor principal stress axes, which facilitates more generalised stress path testing (Fig. 4.18).

In this type of testing, the instrumentation can be attached to the specimen directly, which enables accurate strain measurement. However, some variations in experimental data can be found due to end effects. Also as a result of the specimen geometry (wall curvature) some stress and strain non-uniformity across the specimen can be encountered (particularly when there is a difference between the internal and external confining stress). It has been found by Hight et al. [87], that these problems can be alleviated by having appropriately large ratios of diameter to wall thickness and height to diameter.

The HCA equipment has been used in Nottingham for several years, with the first piece of equipment being developed in 1985. Since then it has been continuously evolved to meet research needs and to gather accurate and reliable data on pavement behaviour under repetitive wheel loading (Figs. 4.19 and 4.20). Outside of Nottingham, Alavi and Monismith [3], have used hollow cylinders to evaluate viscoelastic response characteristics of hot asphalt mixtures under dynamic axial

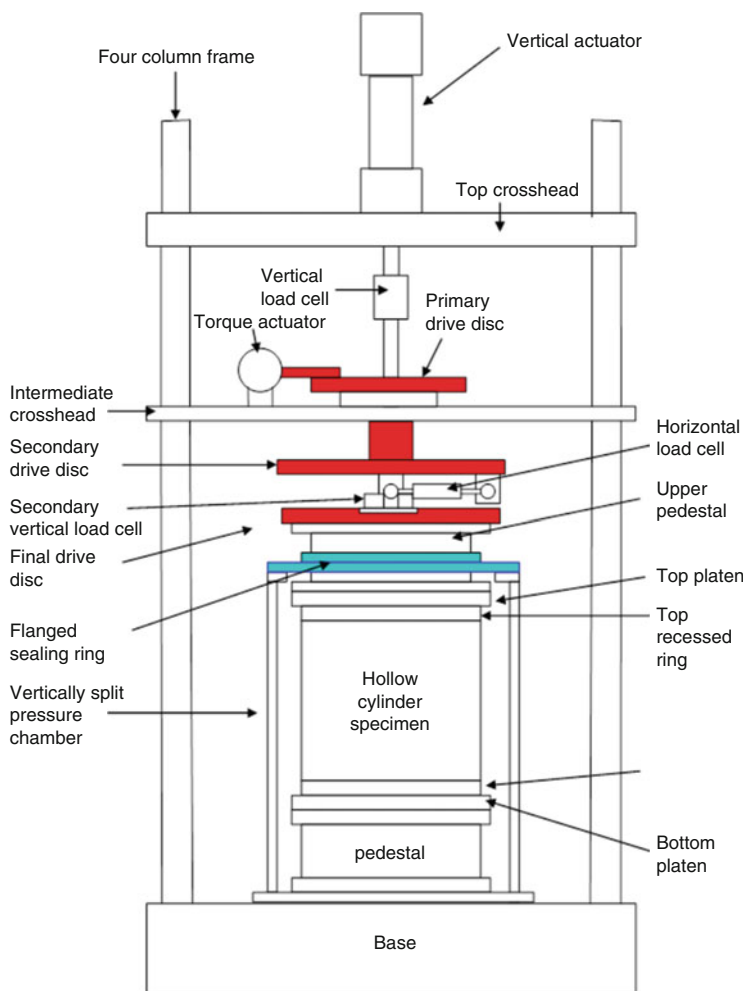


Fig. 4.19 Schematic of the new Nottingham HCA (Tekieli [120])

and shear loads. Elsewhere the HCA has been used to study the effect of principal-plane rotation on permanent deformation in flexible pavements under compressive and shear loading (Crockford [44]).

4.2.2 Non-homogeneous Tests for Material Specimens and Structures

4.2.2.1 Short Overview

The wheel-tracking test (WTT) is used as a simulator of in situ conditions for pavement materials. It is a very simple test and simulative in nature, which has

Fig. 4.20 The HCA loading frame (Tekieli [120])



made it a very popular test for many years throughout the world. Various types of wheel tracker have been developed. These include:

- The French Wheel Tracking Tester (FWTT)
- The Small-Scale UK Wheel Tracking Tester
- The Prototype Small-Scale UK Wheel Tracking Tester
- The Hamburg Wheel Tracking Device (HWTB)
- Superfos Construction Rut Tester
- The Evaluator of Rutting and Stripping (ERSA)
- The Georgia Loaded Wheel Tester (GLWT)
- The Asphalt Pavement Analyser (APA)
- Purdue University Laboratory Wheel Tracking Device (PURWheel)
- Accelerated Pavement Testing (APT)
- Model Mobile Loading Simulator (MMLS)
- Transportek Wheel Tracking Test
- Slab Testing Facilities (STFs)

On a much larger scale, Accelerated Pavement Tests are used to assess specific test sections of asphalt pavement. These will not be treated in this report, but a comprehensive explanation of the different devices can be found in the report of COST 347 (Dawson et al. [49]).

4.2.2.2 Different Types of WTT

In this section the wheel tracking devices will be covered as they appear in the BS EN 12697–22:2003 [27]. This standard classifies devices into four categories. These are extra-large devices, large devices, small devices and small-size devices for use with cores.

Extra-Large Devices

Some requirements are stipulated in the BS EN 12697–22:2003 [27]. The most important of these requirements are listed in the following section. The wheel should be fitted with a 6.00-R9 pneumatic tyre without tread pattern and having a track width of 110 ± 5 mm. The travel of pneumatic tyre relative to the specimen shall be 700 ± 5 mm. The time of travel (outward and return) shall be 2.5 ± 0.5 s. The rolling load applied to the test specimen shall be $10,000 \pm 100$ N at the centre of the test specimen, measured at least when the device is static. The mould(s) should be of internal dimensions 700×500 mm, and capable of withstanding the test conditions without distortion. The equipment shall be set within a ventilated enclosure with a set temperature that is regulated by a probe installed within the test specimen such that the temperature within the specimen is maintained at $\pm 3^\circ\text{C}$ of that set.

Large Devices

Again the requirements for such devices are stipulated in BS EN 12697–22:2003 [27]. The device simulating a rolling load which shall include a wheel fitted with a 400×8 pneumatic tyre without tread pattern and having a track width of 80 ± 5 mm. The pneumatic tyre pressure shall be 600 ± 30 kPa. The travel of pneumatic tyre relative to the specimen shall be 410 ± 5 mm. The frequency of travel (outward and return) shall be 1.0 ± 0.1 Hz.

The rolling load applied to the test specimen shall be $5,000 \pm 50$ N at the centre of the test specimen, measured at least when the device is static. The centre line of the tyre track shall be not more than 5 mm from the theoretical centre of the test specimen. The mould(s) shall have internal dimensions $500 \times 180 \times 50$ mm or $500 \times 180 \times 100$ mm, all dimensions ± 2 mm, capable of withstanding the test conditions without distortion. The device shall be situated in a ventilated enclosure



Fig. 4.21 The French Wheel Tracking Tester (FWTT)

with a set temperature that is regulated by a probe installed within the test specimen such that the temperature within the specimen is maintained at $\pm 2^\circ\text{C}$ of that set.

French Wheel Tracking Tester (FWTT)

The French Wheel Tracking Tester (FWTT) is an example of the type of device that is specified in the standard under the large device section. The French mixture design method uses the FWTT for evaluation of a mixture's resistance to permanent deformation in conditions where the risk of rutting is high.

The FWTT, shown in Fig. 4.21, conforms to the EN 12697-22 [77] standard for large size wheel tracking devices. In this device, two slabs of dimensions $500 \times 180 \times 100$ mm are tested simultaneously. The wheel used to apply the load to the specimen is a 400×8 pneumatic tyre without tread pattern and that has a track width of (80 ± 5) mm. The pneumatic tyre pressure is set to 600 ± 30 kPa. The Trelleborg T522 BV Extra or Special six-ply type pneumatic tyre is suitable for this test. As noted in the research carried out as part of this Task Group, it is important that the six-ply tyre is used and not the weaker two-ply Trelleborg tyre. The travel of pneumatic tyre relative to the specimen shall be 410 ± 5 mm. The frequency of travel (outward and return) is 1.0 ± 0.1 Hz. The rolling load applied to the test specimen is set at $5,000 \pm 50$ N at the centre of the test according to the standard, however it is possible to adjust the load if so required. The angle of skew of the wheel has to be set to $0.0 \pm 0.5^\circ$.

The testing equipment is contained within a temperature controlled chamber, capable of heating up to 60°C. The standard testing temperatures for the testing carried out in these large scale devices is between 50 and 60°C. The temperature is regulated by a probe that is set within the specimen and it has to maintain the temperature of the specimens to within $\pm 2^\circ\text{C}$. The test is run over many cycles. To begin with 1,000 cycles are carried out at ambient temperature. The profile of the specimens is then measured to be the zero point. The profile is measured by measuring the height of the specimen in 15 positions along the wheel path. The values of the 15 measurements are averaged. The specimens are then set to the test temperature and allowed to condition for at least 8 h. The test is then run for either 10,000 or 30,000 cycles. Or run until a rut depth of 10% is exceeded. The rut depth should be measured after 1,000, 3,000, 10,000 and 30,000 cycles, with optional measurements taken at 30, 100, 300 and 100,000 cycles.

Small Devices

As for the other scale devices the requirements are stipulated in BS EN 12697-22:2003 [27]. For the small devices, the wheel-tracking apparatus consists of a loaded wheel that bears on a sample held securely on a table. Either the table or the wheel moves to and from and a device provided monitors the rate at which a rut develops in the surface of the test specimen. Vertical play in the loaded wheel mechanism shall be less than 0.25 mm. The apparatus includes: a tyre of outside diameter between 200 and 205 mm fitted to the wheel. The tyre shall be treadless and have a rectangular cross profile with a width of $w \pm 1$ mm where $w = 50 \pm 5$ mm. The tyre thickness shall be 20 ± 2 mm. The tyre shall be of solid rubber with a hardness number of 80 IRHD units when measured in accordance with ISO 7619 and ISO 48. The wheel load under standard test conditions shall be 700 ± 10 N, measured at the level of the top of the test specimen and normal to the plane of the sample table. The centre of the contact area of the tyre shall describe simple harmonic motion with respect to the centre of the top surface of the test specimen with a total distance of travel of 230 ± 10 mm and a frequency of 26.5 ± 1.0 load cycles per 60 s for the test device. The temperature is controlled such that the temperature of the test specimen during testing is uniform and maintained constant at the specified temperature $\pm 1^\circ\text{C}$. Figure 4.22 shows a schematic view of the wheel-tracking test and Fig. 4.23 shows a picture of a small-scale wheel tracking tester.

The two most common small-scale wheel tracking devices are the UK device to EN 12697-22 [77] small scale standard and the Hamburg Wheel Tracking device.

UK Wheel Tracking Device

The UK device consists of a moving table upon which sits a slab of asphalt $305 \times 305 \times 50$ mm in dimensions contained in a metal mould Fig. 4.23. The table travels backwards and forwards at a rate of 26.5 ± 1.0 cycles per minute.

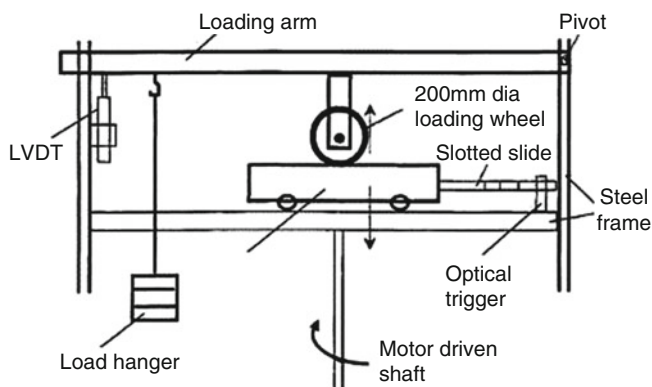


Fig. 4.22 Schematic view of the wheel-tracking test

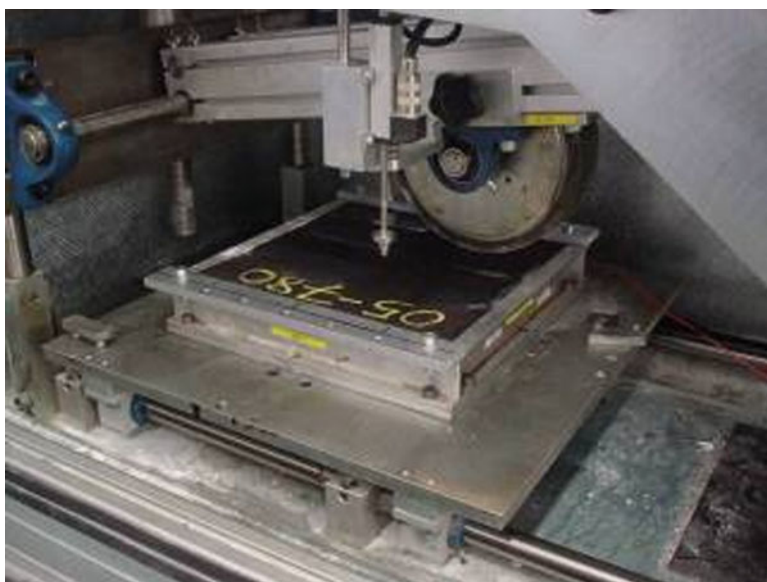


Fig. 4.23 The small-scale UK wheel tracking tester

A wheel with a solid rubber is mounted on an arm which is lowered onto the slab when the test is being run. At the end of the arm a weight is hung to apply the correct loading to the specimen. Wheels with different thicknesses of rubber tyre are used for both the BS 598–110:1998 [24] and the BS EN 12697–22:2003 [27] Wheel tracking standards. Also different loads are also applied. An LVDT is used to measure the rut depth evolution during the test. The testing equipment is housed inside a temperature controlled cabinet, which can be elevated to 60°C. The standard temperature for the testing is either 45 or 60°C. The EN 12697–22 [77]



Fig. 4.24 The small-scale wheel tracking tester

test is carried out over 45 min, but only the rut evolution over the final third of the test is taken into account and the value of rutting is based on the rutting rate for that section of the test. In the BS 598- method, the tracking rate of test specimen is calculated as the rate of increase of track depth T_R , in millimetres per hour (mm/h), from the formulae given in the standard, which are not given here for brevity.

Prototype UK Wheel Tracking Device

At the Nottingham Transportation Engineering Centre, there also exists a prototype wheel tracking device (see Fig. 4.24). This device formed the basis of the UK device, however there are some fundamental differences. Firstly, the load on the prototype device can be varied. A hanger at the end of the loading arm allows the weight to be varied to the users' requirements. Secondly, the size of the moulds is different. The dimensions of slabs that it can take are typically $305 \times 280 \times 100$ mm. The height can be varied anywhere between 50 and 100 mm. Thirdly, the temperature that tests can be carried out at is limited by the temperature controlled room within which the equipment sits. It can be varied from 0 to 40°C, but by the use of inductive heaters it is possible to raise the temperature of the slabs up to 50°C. Finally, the speed of the device can be varied as the mechanism has been fitted with a controller that can vary the speed between approximately 45 passes per minute down to zero. This device has been used for many fundamental pieces of



Fig. 4.25 The Hamburg Wheel Tracking Device (HWTd)

research over the years, including inducing bending forces in beams of asphalt and simulated reflection cracks through overlays (Bodin et al. [15])

Hamburg Wheel Tracking Device (HWTd)

The Hamburg Wheel-Tracking Device (HWTd) measures the combined effects of rutting and moisture damage by rolling a steel wheel across the surface of an asphalt concrete slab that is immersed in hot water. The device was developed in the 1970s by Esso A.G. of Hamburg, Germany, based on the similar British device that had a rubber tyre. The machine was originally called the Esso Wheel-Tracking Device. The City of Hamburg finalised the test method and developed a pass/fail criterion to guarantee that mixtures that pass the test have a very low susceptibility to rutting [123]. This device is shown in Fig. 4.25.

The device was originally used by the City of Hamburg to measure rutting susceptibility. The test was performed for 9,540 wheel passes at either 40 or 50°C. Water was used to obtain the required test temperature instead of an environmental air chamber. The number of wheel passes was later increased to 19,200 and some mixtures began to deteriorate from moisture damage. Greater than 10,000 wheel passes was generally needed to show the effects of moisture damage.

The machine tests slabs that typically have a length of 320 mm, a width of 260 mm, and a thickness of either 40, 80, or 120 mm. Thicknesses up to 150 mm can be tested. The thickness of the slab is specified to be a minimum of three times the nominal maximum aggregate size. The mass of a slab having a thickness of

80 mm is approximately 15 kg. Pavement cores having a minimum diameter of 250 mm can also be tested.

The required air-void level for laboratory-prepared specimens is not given by the City of Hamburg procedure. The Federal Highway Administration at the Turner-Fairbank Highway Research Center is using $7 \pm 1\%$ air voids for dense-graded hot-mix asphalts, and $5.5 \pm 0.5\%$ for stone matrix asphalts. The Colorado Department of Transportation (CDOT) also uses $7 \pm 1\%$ air voids for dense-graded hot-mix asphalts (Aschenbrener [7]).

Specimens are secured in reusable steel containers using plaster of Paris. Each specimen is placed into a container so that its surface is level with the top edge of the container. This allows the full range of the rut depth measurement system to be utilised. Containers are manufactured in heights of 40, 80, and 120 mm. Steel spacers can be placed under cores and pavement slabs if needed. The container with the specimen is then placed into the wheel tracking device. The container rests on steel; this provides a rigid, load-bearing base for the specimen.

The temperature of the water bath can be set from 25 to 70°C. The most commonly used test temperature in Hamburg is 50°C, although 40°C has been used when testing certain base mixtures. A water temperature of 50°C is reached within 45 min. Specimens are conditioned at the test temperature for a minimum of 30 min. Heat is provided by heated coils in the water. The temperature of the water is then maintained by these heating coils and by introducing cold water from a tap.

The device tests two slabs simultaneously using two reciprocating solid steel wheels. The wheels have a diameter of 203.5 mm and a width of 47.0 mm. The load is fixed at 685 N and the average contact stress given by the manufacturer is 0.73 MPa. This assumes an average contact area of 970 mm², which is based on the 47.0 mm wheel width and an average contact length of 20.6 mm in the direction of travel. However, the contact area increases with rut depth, and thus the contact stress is variable. The manufacturer states that a contact stress of 0.73 MPa approximates the stress produced by one rear tyre of a double-axle truck. The average speed of each wheel is approximately 1.1 km/h; each wheel travels approximately 230 mm before reversing direction, and the device operates at approximately 53 ± 2 wheel passes/min.

Superfos Construction Rut Tester (SRCT)

A slight modification was made to the Hamburg Wheel Tracking Tester by the Superfos Construction Company. This device is known as the Superfos Construction Rut Tester (SCRT). The slab specimens tested remained the same, but the loading mechanism was changed. The SCRT applies an 82.6 kg vertical load onto a solid rubber wheel with a diameter of 194 mm and a width of 46 mm. This configuration results in a contact pressure of 940 kPa and a contact area of 8.26 cm². The loading is applied at a speed of approximately 556 mm per second. Test temperatures used with this device vary between 45 and 60°C. The results obtained seem similar to those obtained with the HWTD, with a similar creep slope/stripping slope behaviour and stripping inflection point (Cooley et al. [43]).

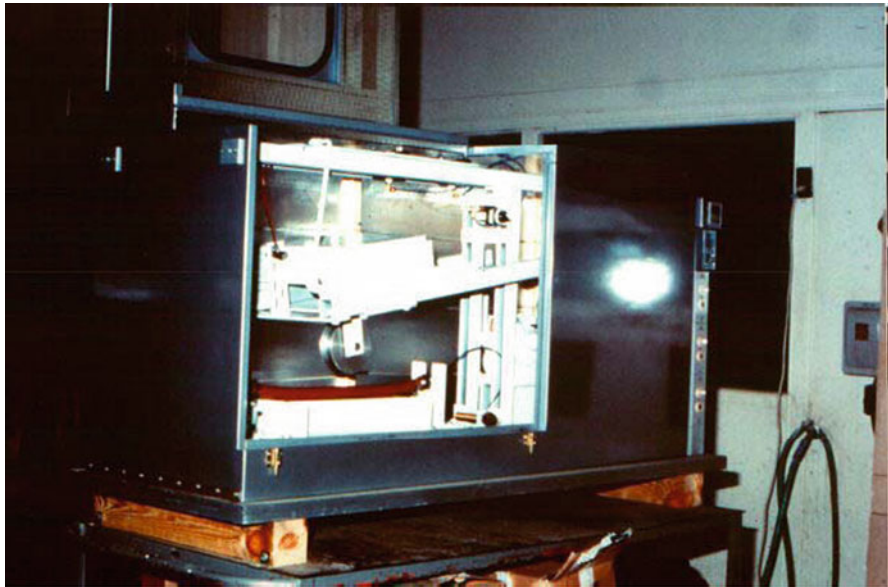


Fig. 4.26 The Georgia Loaded Wheel Tester (GLWT)

Evaluator of Rutting and Stripping (ERSA)

A further modification was carried out on the HWTD by the Department of Civil Engineering at the University of Arkansas, and is known as the Evaluator of Rutting and Stripping (ERSA). This device tests cylindrical or beam specimens and can be conducted in either wet or dry conditions. A 47 mm wide steel wheel is used to load specimens with 705 N for 20,000 cycles or a 20 mm rut depth, whichever occurs first (Cooley et al. [43]).

Georgia Loaded Wheel Tester (GLWT)

The Georgia Loaded Wheel Tester (GLWT) developed by the Georgia Department of Transportation has been used since 1985 during the design stage to evaluate rutting susceptibility of asphalt concrete mixtures (Fig. 4.26). In the GLWT testing, asphaltic concrete is subjected to an elevated temperature in a loaded wheel system under repetitive loading conditions, and the permanent deformation induced under the wheel path is measured. This approach to assess rutting susceptibility was thought to be much more representative than the current test methods. It can also provide a fast and more accurate means of assessing rutting susceptibility of asphalt concrete under actual field conditions. The asphaltic beam samples used for the GLWT testing are prepared by a rolling compaction machine. To promote the concept of using the GLWT as a supplement to the Superpave Level 1 design procedure for evaluating permanent deformation of hot-mix asphalt (HMA), a test method utilising the Superpave gyratory compactor-prepared samples to evaluate

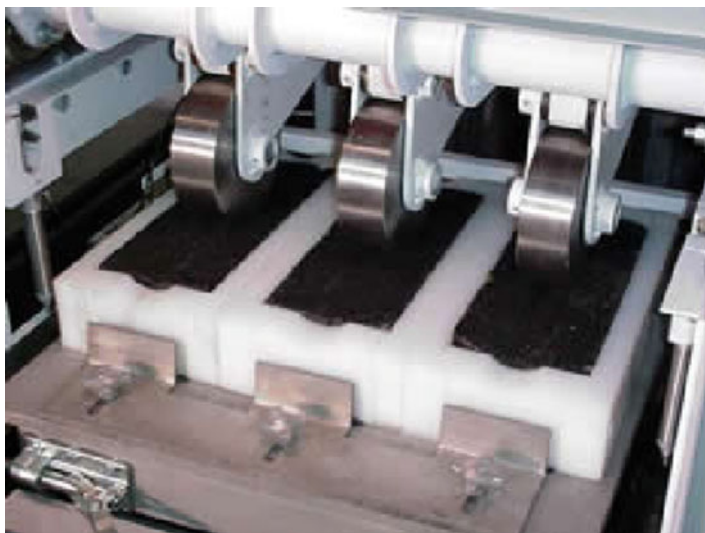


Fig. 4.27 The Asphalt Pavement Analyzer (APA)

the rutting resistance of HMA by the GLWT was developed. Samples can be tested at either 40, 50 and 60°C for 8,000 cycles under the standard GLWT testing procedure (Collins et al. [40]).

Asphalt Pavement Analyzer (APA)

The Asphalt Pavement Analyzer (APA) is a multifunctional Loaded Wheel Tester (LWT) used for evaluating permanent deformation (rutting), fatigue cracking and moisture susceptibility of both hot and cold asphalt mixtures (Fig. 4.27). Testing time for a complete permanent deformation evaluation is 2 h and 15 min (8,000 cycles). The testing time for fatigue cracking evaluation is dependent upon the fatigue behaviour of the mix being evaluated. Permanent deformation (rutting) susceptibility of mixes is assessed by placing beam or cylindrical samples under repetitive wheel loads and measuring the amount of permanent deformation under the wheel path. The APA features an Automated Data Acquisition System, which obtains rutting measurements and displays these measurements in a numeric and graphical format. Five measurements can be taken during a single pass over a beam specimen and two measurements can be taken during a single pass over a cylindrical specimen. The APA features controllable wheel load and contact pressure that are representative of actual field conditions. Each sample can be subjected to a different load level (up to 113 kg) resulting in Contact Pressures up to 1,378 kPa. Triplicate beam samples, or six cylindrical (gyratory, vibratory, Marshall pills, roadway cores) samples in three specially designed sample moulds can be tested under controllable temperature and in dry or submerged in water environments. Fatigue cracking resistance of asphalt concrete can be determined by subjecting

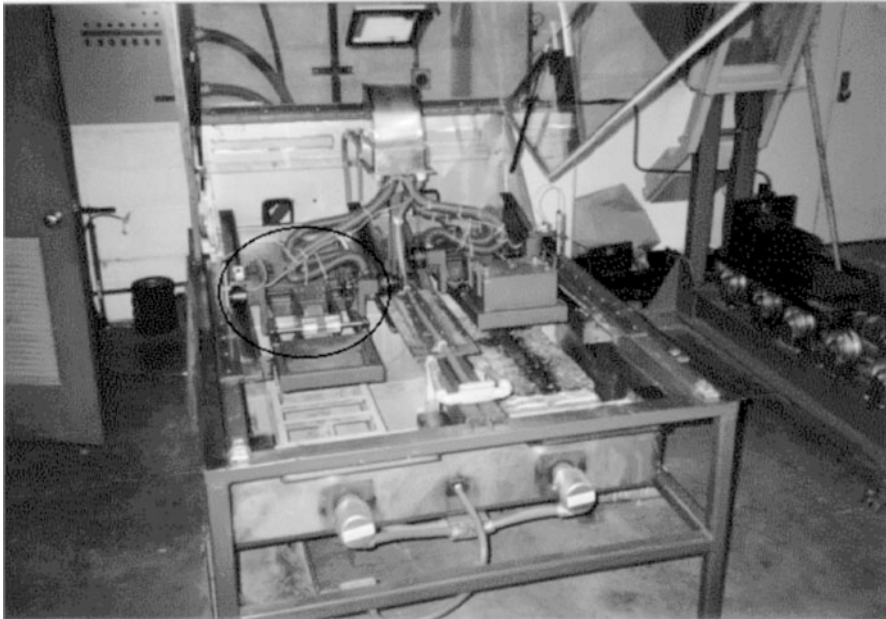


Fig. 4.28 The Purdue University Laboratory Wheel tracking device (PURWheel)

beam samples to a repeated wheel load of controllable magnitude and contact pressure in a low temperature environment (Cooley et al. [43]).

Purdue University Laboratory Wheel Tracking Device (PURWheel)

This device was developed at Purdue University. Specimens used in conjunction with this device are either cut from the roadway or compacted in the laboratory. Slab specimens are 290 mm wide by 310 mm long (Fig. 4.28). The thicknesses of the specimens depend on the mixture type being tested. For surface courses a sample thickness of 38 mm is used. In binder and base course mixtures, 51 and 76 mm are used respectively.

PURWheel was designed to evaluate rutting potential and/or moisture sensitivity. Test samples can be tested in both dry and wet conditions. Moisture sensitivity is defined as the ratio of the number of cycles to 12.7 mm of rutting in wet conditions over the number of cycles to 12.7 mm in dry conditions. Cycles to 12.7 mm the rut depth are used to distinguish between good and bad performing mixtures with respect to rutting.

Loading is applied by means of a pneumatic tyre. A gross contact pressure of 620 kPa is applied to the sample. This is carried out by applying a load of 175 kg onto the wheel that is pressurised to 793 kPa. A loading rate of 332 mm/s is applied and the test is run until 20,000 wheel passes or 20 mm rutting has been accumulated.

PURWheel is very similar to the HWTD, however it can incorporate wheel wander, which is a unique feature amongst Loaded Wheel Testers that are common within the USA (Cooley et al. [43]).

Small-Size Devices for Use with Cores

According to BS EN 12697–22:2003 [27], some of the small devices mentioned above can be used in conjunction with cored samples. An abrasive disc circular saw is used to cut through cores of compacted bituminous mixture up to 300 mm in diameter to provide a nominally ridge-free surface. A core clamping assembly is used, comprising two steel, glass or other suitably stiff plates, and clamping blocks for holding the prepared core in place whilst it is bedded in the holding medium. The holding medium used can be either plaster of Paris or a polyester resin and hardener. A spirit level, at least 100 mm long, is used to make sure the core is set parallel to the wheel motion.

Accelerated Pavement Testing (APT)

There are several institutions which have their own accelerated pavement testing facility. Some of these are classed as full-scale, which can apply a standard wheel load to a laboratory built pavement. An example of such a device which is full-scale is the APT at the Transport Research Laboratory (TRL) in the UK. Others, such as the device at the Nottingham Transportation Engineering Centre, are classed as half-scale as they can only apply half of a standard axle loading. Other devices include LINTRACK at TUDelft, The Danish Road Testing Machine (RTM) in Lyngby. A full list of such APT devices can be found in the final report of COST 347 (Dawson et al. [49]).

Model Mobile Load Simulator (MMLS)

The MMLS Mark 3 is a wheel tracking device that can be used both in the laboratory, on slabs or cylindrical specimen, and in the field (Figs. 4.29 and 4.30). The machine has four bogies with 300 mm diameter pneumatic tyred wheels, operating uni-directionally. The typical speed used by commercial laboratories for testing is 7,200 repetitions per hour. The maximum achievable tire inflation pressure is 800 kPa and the maximum achievable load is 2.9 kN. Load conditions and speed can be varied, but the majority of tests have been performed at a 2.9 kN load level with a 750 kPa tyre inflation pressure (Cooley et al. [43]).

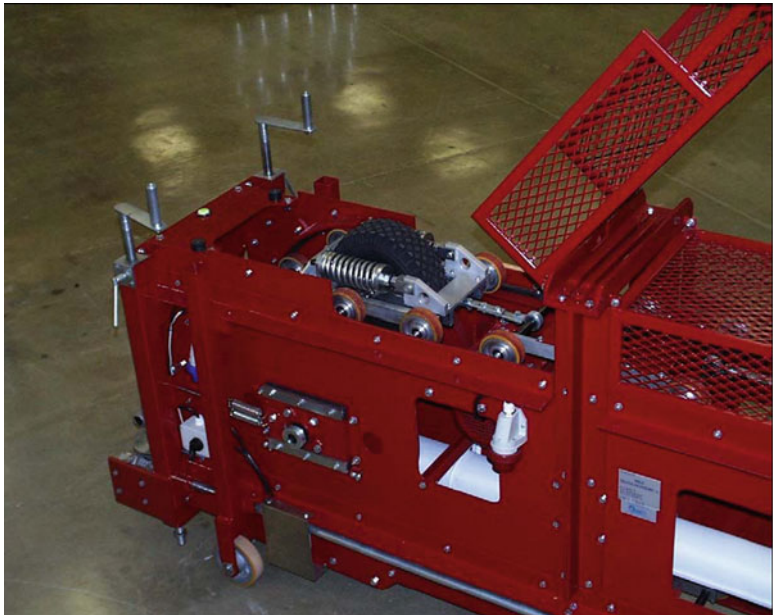


Fig. 4.29 The Model Mobile Load Simulator (*MMLS*)

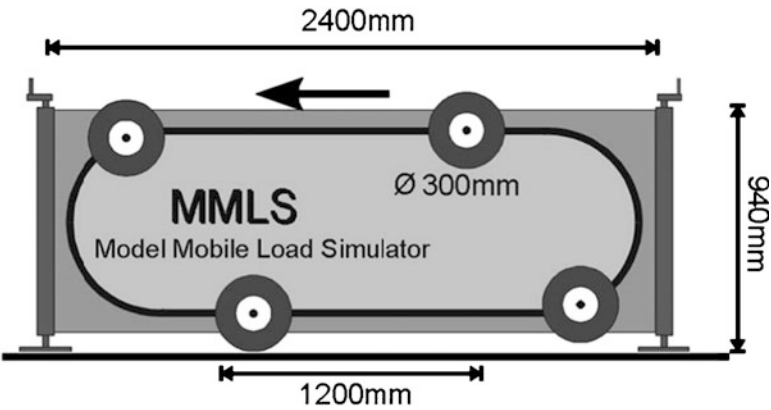


Fig. 4.30 Operation principle of a mobile load simulator, exemplified with a schematic view of the MMLS

Transportek Wheel Tracking Test (TWTT)

The Transportek Wheel Tracking Test (TWTT), shown in Fig. 4.31, is an adaptation of the TRL slab compactor. The equipment can be used to compact 655 mm × 345 mm slabs using a segment of a steel wheel roller. The resulting compaction action resembles the compaction effort in the field. The machine can be transformed



Fig. 4.31 The Transportek Wheel Tracking Tester (TWTT)

to evaluate rutting potential of HMA by means of wheel tracking with a loaded solid rubber wheel. The wheel diameter is 400 mm with a width of 100 mm. The contact stress can be varied from 600 to 900 kPa, which is achieved by varying the load. The temperature is controlled by means of a temperature cabinet. The speed of the wheel can be adjusted, but is typically set at 2,500 passes per hour. Tests described in Long and Verhaeghe [94] were performed at a standard 900 kPa contact stress generated by a 6 kN load.

Slab Testing Facilities (STF)

Several institutions have their own slab facilities. These tend to be large prototype devices, which are built specifically for the laboratory in question. Such devices exist or have existed at Finland, NTEC in the UK and the University of Ulster in the UK.

The slab testing facility (STF) at NTEC can be used to test both compacted asphalt specimens and also compacted granular material. The carriage is mounted onto a pair of beams and is driven by means of a wire rope tensioned around a drum, which is axially coupled to the motor. The loading is applied via a hydraulic actuator located at one end of the facility. A feedback mechanism is used which involves the use of four load cells located under the corners of the steel pallet. A constant wheel load can only be achieved by applying an increasing actuator load of the correct gradient as the wheel approaches the actuator. Alternatively, the

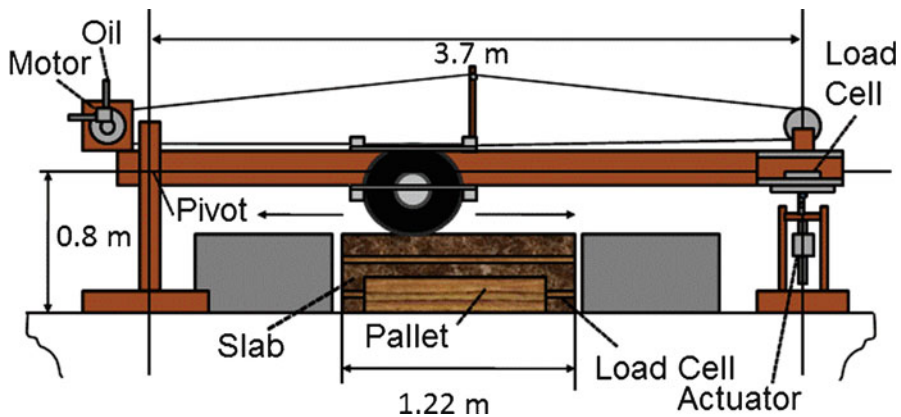


Fig. 4.32 Schematic of the NTEC Slab Testing Facility (SFT)



Fig. 4.33 The pavement test facility at University of Oulu in Finland

wheel can be fixed at a position directly above the compacted material. A repeated vertical load can then be applied via the hydraulic actuator, regulated by the means of a signal generator.

The dimensions of the slabs used within the facility are approximately 1.2 m long by 1 m wide and 100 mm in height. A maximum of 5 kN load could be applied and the machine operated at speeds of up to 20 passes per minute. The specimens could be tested until a 20 mm maximum deformation was reached. Figure 4.32 shows a schematic of the facility (Hughes [91]; Chan [37]).

The Pavement Test Facility University of Oulu in Finland is a slab testing facility (see Figs. 4.33 and 4.34). It has a pavement length of 1.2 m and a useful

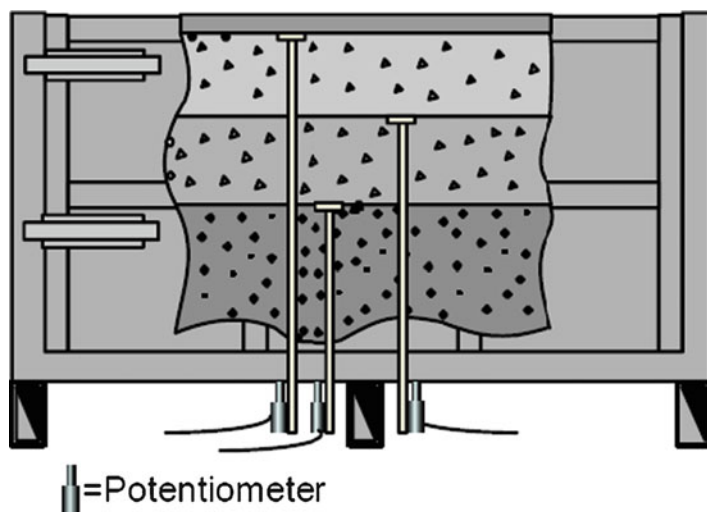


Fig. 4.34 Schematic of the instrumentation of the pavement test facility at University of Oulu in Finland

testing length 0.4 m. Besides being 1.2 m in length, the slabs are 600 mm thick and 900 mm wide. The loading range that can be applied is 14–50 kN. It operates at a constant speed of 5 km/h meaning it can perform 2,000 passes/h at a constant speed. The wheels loads (half axle) are converted to the corresponding axle loads (Dawson et al. [49]).

4.3 Wheel Tracking Tests (WTT) Campaigns on Mixtures Made with Pure and Modified Bitumen

As presented previously, different devices were developed to reproduce the effect of a wheel rolling over the surface of a bituminous layer. Among these devices, the most used is the Wheel Tracking Tester (WTT). Specimen sizes, test conditions (load, frequency, temperature, etc.) and type of wheel could be very different Between WTT device types (see Sect. 4.2.2). The backwards and forwards motion of a loaded wheel passing on the top surface of a bituminous specimen gives an evaluation of the rutting performance of the tested material. The bituminous mixture specimen could be prepared in the laboratory or cut from a road site. During the test, the rut is expressed according to the number of wheel cycles. WTT test results are usually analysed as a pass/fail test that qualifies the studied material.

In the framework of Working Group 3 of the RILEM Technical Committee 206 “Advanced Testing of Bituminous materials” (ATB), two experimental campaigns were organized to check the ability of wheel tracking tests. These two campaigns

are described and analysed in the next sections. The first campaign “campaign A” considers bituminous mixtures made with pure bitumen. The mixtures are tested with different types of WTT. For the second campaign “campaign B” only the French Wheel tracking test (FWTT) is used to characterise a Polymer modified Bitumen (PmB). These campaigns are introduced in the papers by Perraton et al. [106] and Gabet et al. [81].

The members of the RILEM Technical Committee 206-ATB who worked for the two campaigns as well as the laboratories and their technical and scientific staff, which participated in the test programme, are gratefully thanked for their efficient contribution.

4.3.1 Wheel Tracking Test Campaign “A” on Pure Bitumen Mixtures

4.3.1.1 Introduction

The RILEM ATB testing program of the campaign A on mixtures made with pure bitumens was planned in order to:

1. Investigate suitability of WTT for evaluating rutting performance onsite for different bituminous pavement systems;
2. Establish reproducibility of the French wheel tracking tester (FWTT);
3. Evaluate the response of different types of testing devices;
4. Determine the effect of testing temperature.

Laboratories located in seven different countries, were involved in this testing programme and are identified by roman numerals from I to VII, in the next sections. The seven participating laboratories are: Centro del Estudios Transporte (Spain), EMPA (Switzerland), IBDIM (Poland), LCPC (France), LUCREB (École technologie supérieure, Québec, Canada), NTEC (University of Nottingham, England) and University of Palermo (Italy).

Three bituminous layer systems were selected (A, B and C) to be tested. Bituminous layer systems are composed of two or three different bituminous mixtures having different thicknesses. Bituminous systems A and B were sawn from two road sites in Switzerland. Bituminous system C was extracted from the circular test track of LCPC in Nantes (France). The location of road sites where bituminous systems A and B are used in Switzerland is indicated in Fig. 4.35.

4.3.1.2 Testing Program

Specimen Preparation

The tested specimens were cut from large sawn slabs extracted from the on-site structures. EMPA took charge of operations for structures A and B while LCPC

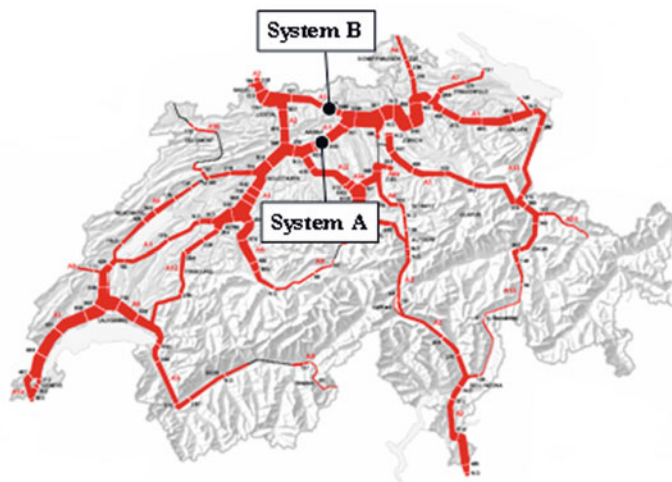


Fig. 4.35 Location of road where tested bituminous systems A and B are used in Switzerland

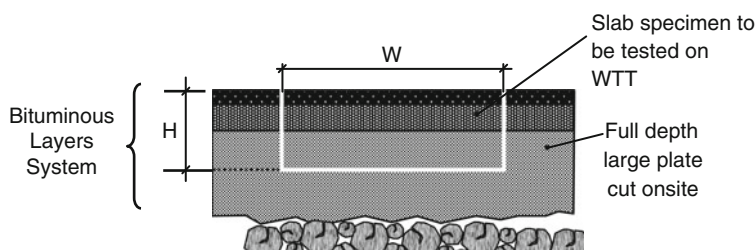


Fig. 4.36 Schematic view of the full depth large plate cut onsite and trimmed slab specimen shipped to each team to be tested with WTT (Width, Length and Height)

dealt with structure C. Large slabs cut on-site were trimmed in the laboratory to meet sample slab size requirements for laboratory testing (Fig. 4.36). Slab dimension requirements are presented in Table 4.1. Slab specimen cutting was done in a way to keep the top surface of the bituminous layer system intact. A set of two slabs per series of tests were shipped to each participant of the laboratory test program.

The different bituminous layer systems of the full depth large plate cut on-site are presented in Fig. 4.37.

Mix Design of Bituminous Mixtures

The mix designs of each bituminous layer are presented in Table 4.2, while the specific aggregate grading curves are plotted in Fig. 4.38. Figure 4.38d shows a comparison of the aggregate grading of the top layers of the three bituminous systems.

Table 4.1 Characteristics of the three WTT devices used

WTT Devices		FWTT	WTT-I	WTT-II
Standard	Number	EN12697-22	EN12697-22	Close to the EN12697-22
Loading wheel system	Procedure	Procedure B, in air		
	Diameter (mm)	200 ± 5	200 ± 5	240 ± 5
	Width (mm)	80 ± 5	50 ± 5	50 ± 5
	Contact surface (mm ²)	~7,815	~1,000	~1,000
	Travel (mm)	410 ± 5	230 ± 5	230 ± 5
	Tyre	Solid rubber	Not applicable	Hardness: 80 IRHD (thickness : 20 ± 2)
Loading	Surface Pressure	Smooth surface, without groove	Not applicable	Not applicable
		6 ± 0.3 bars (at the beginning)		
	Live load applied (N)	5,000 ± 50	700 ± 10	880 ± 10
	Wheel cycle (Hz)	1	0.43	0.35
Tested slab	Loading cycle (Hz)	2	0.87	0.70
	Size: W, L, H (mm ³)	100, 500, H = 100 mm	300, 300, H = 50–100 mm	
	Fixation in the mould	Plaster of Paris applied on edges and base ^a		
	Rut calculation	[rut (mm)/H(mm)] × 100%		
		Points of measurement	15	rut is measured
on a continuously process (LVDT) ³ ^a Required for sample taken from road site				

SYSTEM «A» SMA 11S		SYSTEM «B» MA 11		SYSTEM «C» BBSG 0/14	
42 mm	Top Layer: SMA 11 Binder: 6.80%	45 mm	Top Layer: MA 11 Binder: 6.70%	60-70 mm	Top Layer: BBSG0/14 Binder: 5.37%
80 mm	Base Layer: AC 22 Binder: 4.30%	36 mm	Bond Layer: MA16 Binder: 6.70%	80 mm	Base Layer: Grave bitume– Class 3
		73 mm	Base Layer: AC 22 Binder: 4.30%		Binder: 4.80%

Fig. 4.37 Bituminous layer systems A, B and C of the full depth large plate cut on-site

Description of Wheel Tracking Test Devices

The laboratory investigation was conducted on three different types of WTT devices. Figure 4.39 shows pictures of the three testing devices and Table 4.1 gives the main characteristics of the devices used. As indicated in Table 4.1, the three testing devices consider different specimen sizes, tyre types and loading frequencies. Two slabs are tested at the same time (left and right) with the FWTT whereas the two other devices test only one slab at the time.

Table 4.2 Mix design proportions of the bituminous layers of each tested systems

System	Layer	Thickness (mm)	Type	Bitumen		
				b% ^a	Pen	T _{BA} ^b (°C)
A	Top	42	SMA 11 S ^c Stone Mastic Asphalt: SMA	6.80	35	60.2
	Base	80	AC 22 ^c Asphalt Concrete : AC	4.30	13	71.6
B	Top	45	MA 11 ^c Mastic Asphalt: MA	6.70	31	63.4
	Bonding	36	MA 16 ^c	6.70	35	62.8
	Base	73	AC 22 ^c	4.50	34	60.0
C	Top	60–70	BBSG 0/14 ^d Béton bitumineux semi grenu : BBSG	5.37	50/70	–
	Base	80	GB 0/14 ^d Grave Bitume: GB	4.80	50/50	–

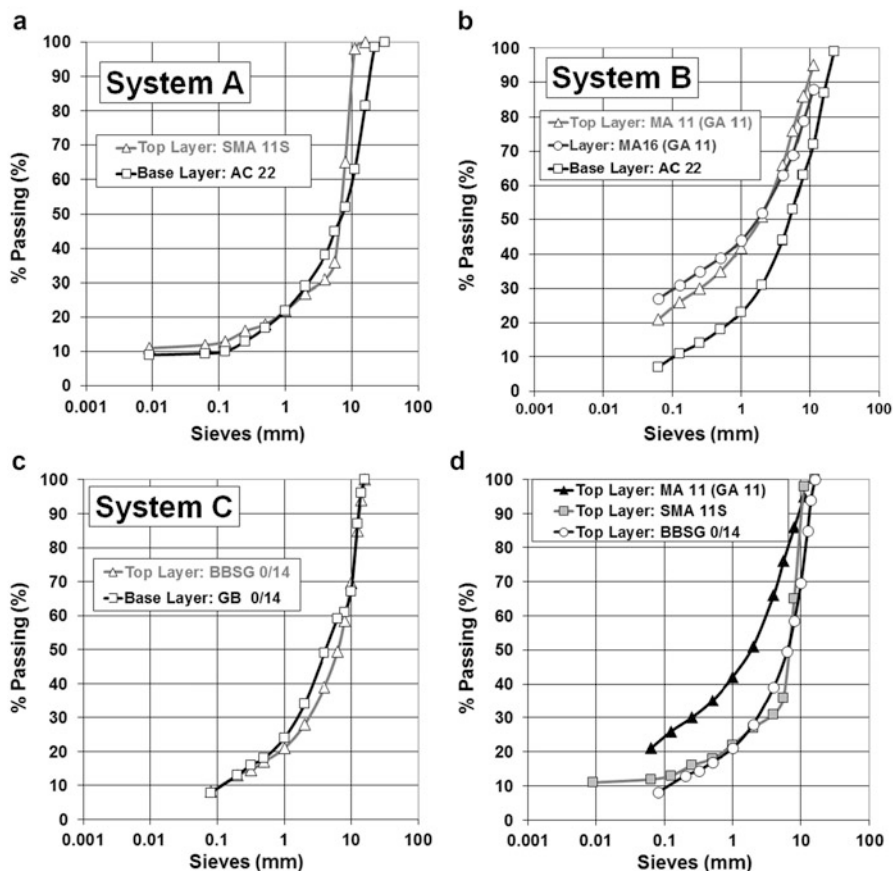
^a Bitumen content is express in accordance to the weight of aggregate^b T_{BA}: ring and ball temperature^c Swiss standard^d French Standard**Fig. 4.38** Grading curves of aggregate used for bituminous materials: (a) System A, (b) System B, (c) System C and (d) Grading curves of aggregate of the *top* layers of the three bituminous tested systems



Fig. 4.39 Pictures of the three WTT devices used in the experimental programme. (a) FWTT (large size device: EN12697-22), (b) WTT-I (Small size device: EN12697-22) and (c) WTT-II (Small size device: close to the EN12697-22)

Table 4.3 Testing program performed by each laboratory

Laboratory	Testing temperature (°C)	WTT type
I	50 and 60	FWTT
II	50	
III	60	
IV	60	
V	50 and 60	
VI	50 and 60	WTT-I
VII	60	WTT-II

N corresponds to the number of wheel cycles (FWTT: wheel cycle of 1 Hz) and, a wheel cycle corresponds to two passes at the centre of the tested slab (FWTT: loading cycle of 2 Hz).

For the FWTT, the rut is measured manually at 15 different locations on the top surface of a tested slab for different values of N (30; 100; 300; 1,000; 3,000; 10,000 and 30,000 cycles). Consequently, for each measurement step, the wheel motion is stopped and the cabinet is opened. The wheel motion is restarted after the temperature measured within the slab reaches the required temperature. The control temperature is measured with a temperature probe placed in the right corner of the left slab.

For the WTT-I device, the rut is measured continuously with an LVDT. The testing time is about six and half hours, which correspond, to 10,000 cycles.

For the WTT-II device, the rut is measured at three different locations on the top surface for different values of N (0; 42; 126; 210; 420; 630; 840; 1,050; 1,260; 1,470; 1,680; 1,890; 2,520; 3,150; 3,780; 4,410 and 5,040 cycles). The current testing time is 120 min, which corresponds to 5,040 cycles.

For all WTT devices, the rutting test is performed in a temperature-controlled cabinet, with possible temperature settings ranging between 50 and 65°C. During this experimental program, two testing temperatures were investigated: 50 and 60°C. Table 4.3 shows the temperature considered by each laboratory (I–VII).

Specimen Preparation

Shipped bituminous slab specimens already cut at the right dimensions are fixed in the steel mould of the device using plaster of “Paris”. The plaster of “Paris” was bedded into the mould in order to achieve satisfactory contact between the specimen and the bottom/side walls of the mould as shown in Fig. 4.40.

Rut Determination

The rutting performance of the three bituminous layer systems was compared from laboratory measurements to onsite measurement or observation.



Fig. 4.40 Specimen preparation for WTT testing

In the laboratory, the rut was measured after chosen cycle intervals in the wheel path by measuring the settlement of the top surface of the specimen. These measurements are reported in millimetres and the rut is expressed in percentage according to the total thickness of the bituminous tested slab. The rut is expressed as follows:

$$\text{rut}(\%) = \frac{\text{Top surface settlement (mm)}}{\text{Total thickness of bituminous tested slab (mm)}} \bullet 100\% \quad (4.9)$$

For on-site measurement, the rut was also measured at chosen cycles (50,000; 100,000 and after every 100,000 cycles up to 15,000,000) in the wheel path by measuring the road deflection. These measurements are reported in millimetres and the rut is expressed in percentage according to the total thickness of bituminous layers of the pavement structure. The rut is expressed as follows:

$$\text{rut}(\%) = \frac{\text{Top surface deflection (mm)}}{\text{Total thickness of bituminous layers of the pavement (mm)}} \bullet 100\% \quad (4.10)$$

4.3.1.3 French Wheel Tracking Test (FWTT) Results

As shown in Table 4.3, five countries used the FWTT device. Rutting data from each participant is presented in Tables 4.4 and 4.5.

Temperature Evolution in Slab During Test

Under cyclic wheel loading, the temperature in bituminous material specimen rises due to viscous energy dissipation inside the material and tyre-specimen interaction at the interface. During the WTT test, the temperature of the bituminous material must be properly controlled and maintained in a certain range given by standard.

Evolutions of the temperature (T-slab 1 and T-slab 2) in the two tested slabs under testing from laboratory I are presented in Fig. 4.41 for the three bituminous

Table 4.4 FWTT results for all system (A, B and C) tested at 60°C

Laboratory														
I		V		IV		III			n	t _{0.975}	$\bar{x}$	s	X ⁺	X ⁻
1	2	1	2	1	2	1	2							
(a) System A (SMA 11S)														
N	30	1.18	1.94	0.88	1.56	1.78	1.61	1.49	6	2.49	1.49	0.36	1.89	1.09
	100	1.83	2.53	1.38	2.32	2.33	2.29	2.11	6	2.49	2.11	0.39	2.54	1.68
	300	2.51	3.05	1.80	3.25	2.88	2.79	2.71	6	2.49	2.71	0.47	3.24	2.19
	1,000	3.65	3.81	2.52	4.38	3.58	3.15	3.52	8	2.37	3.52	0.58	4.04	3.00
	3,000	4.77	4.49	3.37	5.96	4.20	4.20	4.48	8	2.37	4.48	0.74	5.14	3.82
	10,000	5.56	5.12	4.57	7.33	5.00	5.12	5.35	8	2.37	5.35	0.87	6.12	4.57
	30,000	6.50	5.72	5.15	7.85	5.59	5.69	6.05	8	2.37	6.05	0.85	6.81	5.29
(b) System B (GA 11 T)														
N	30	2.77	3.31	2.72	3.10	3.83	3.92	2.49	4	6	2.49	3.27	0.47	3.80
	100	6.16	6.33	5.03	5.97	7.72	8.03	2.49	4	6	2.49	6.54	1.03	7.69
	300	11.42	10.40	9.47	10.66	14.16	14.33	2.37	4	8	2.37	11.88	1.66	13.36
	1,000	19.24	18.08		17.27	17.69	17.69	3.18	4	4	3.18	18.07	0.73	19.42
Laboratory														
I		V		IV		III			n	t _{0.975}	$\bar{x}$	s	X ⁺	X ⁻
1	2	1	2	1	2	1	2							
(c) System C (BBSG)														
N	30	1.03	1.11	0.77	1.10	1.28	1.13	1.07	6	2.49	1.07	0.15	1.24	0.90
	100	1.53	1.64	1.04	1.59	1.70	1.74	1.54	6	2.49	1.54	0.23	1.80	1.28
	300	2.03	2.29	1.29	2.08	2.47	2.47	2.11	6	2.49	2.11	0.40	2.55	1.66
	1,000	2.70	3.18	2.44	2.88	3.23	3.31	2.39	10	2.26	2.39	0.80	2.99	1.79
	3,000	3.54	4.59	2.99	3.32	3.99	4.24	3.09	10	2.26	3.09	1.00	3.84	2.34
	10,000	4.29	6.09	3.68	4.04	4.96	5.34	3.90	10	2.26	3.90	1.25	4.84	2.96
	30,000	5.21	7.99	4.37	4.70	6.11	6.66	5.10	10	2.26	5.10	1.36	6.12	4.07

Table 4.5 FWTT results for all system (A, B and C) tested at 50°C

		Laboratory									
		I		V							
		1	2	1	2	n	t _{0.975}	$\bar{x}$	s	X ⁺	X ⁻
(a) System A (SMA 11S)											
N	30	0.99	1.23	0.74	0.88	4	3.18	0.96	0.18	1.29	0.63
	100	1.54	1.80	1.20	1.28	4	3.18	1.45	0.24	1.89	1.02
	300	2.01	2.18	1.43	1.57	4	3.18	1.80	0.31	2.36	1.23
	1,000	2.32	2.44	1.78	1.94	4	3.18	2.12	0.27	2.62	1.62
	3,000	2.87	3.27	2.17	2.36	4	3.18	2.67	0.43	3.46	1.88
	10,000	3.84	3.98	2.75	2.95	4	3.18	3.38	0.54	4.37	2.40
	30,000	4.29	4.71	2.89	3.09	4	3.18	3.75	0.77	5.16	2.33
(b) System B (GA 11T)											
N	30	1.56	1.97	1.10	0.98	4	3.18	1.40	0.39	2.12	0.68
	100	3.17	3.45	2.03	1.87	4	3.18	2.63	0.69	3.90	1.36
	300	5.32	5.55	3.29	3.35	4	3.18	4.38	1.06	6.33	2.43
	1,000	9.67	9.25	4.99	6.12	4	3.18	7.51	2.00	11.18	3.84
		Laboratory									
		I		V							
		1	2	1	2	n	t _{0.975}	$\bar{x}$	s	X ⁺	$E_0^{GM} = \left(\sum_{i=0}^n \frac{1}{E_i^{GMV}} \right)^{-1}$
(c) System C (BBSG)											
N	30	1	2	1	2	4	3.18	0.43	0.10	0.61	0.25
	100	0.39	0.30	0.57	0.45	4	3.18	0.74	0.16	1.03	0.45
	300	0.66	0.55	0.97	0.78	4	3.18	1.13	0.19	1.48	0.78
	1,000	1.11	0.92	1.44	1.04	4	3.18	1.61	0.31	2.18	1.04
	3,000	1.65	1.27	2.10	1.44	4	3.18	2.37	0.55	3.38	1.35
	10,000	2.61	1.93	3.16	1.77	4	3.18	3.40	0.97	5.18	1.61
	30,000	3.52	3.06	4.85	2.15	4	3.18	4.11	1.24	6.38	1.83

layer systems (A, B and C) tested at 60°C. Temperatures were recorded at each rut measurement (Rut Slab 1 and Rut slab 2) of the two tested slab, just before opening the doors of the cabinet. The temperature is always inside of the acceptable temperature limits to perform the test (58–62°C). Note that for System B, due to the poor performance of materials the test was stopped after less than 1,000 cycles.

Sample Mean and Interval of the Population Mean

As indicated in Tables 4.4 and 4.5, many repetitions (4–8) were performed for the three tested bituminous layer systems (A, B and C) using the FWTT device. The overall average values obtained for each tested system ($\bar{x}_A$, $\bar{x}_B$ and $\bar{x}_C$) are designated hereafter as the sample mean.

It is important to differentiate between the population mean (μ) and the sample mean ($\bar{x}$). The population mean represents the true value when an infinite number of measurements are made (Baillargeon and Rainville [11]). The sample mean refers to the average value of a series of measurements to estimate the population mean.

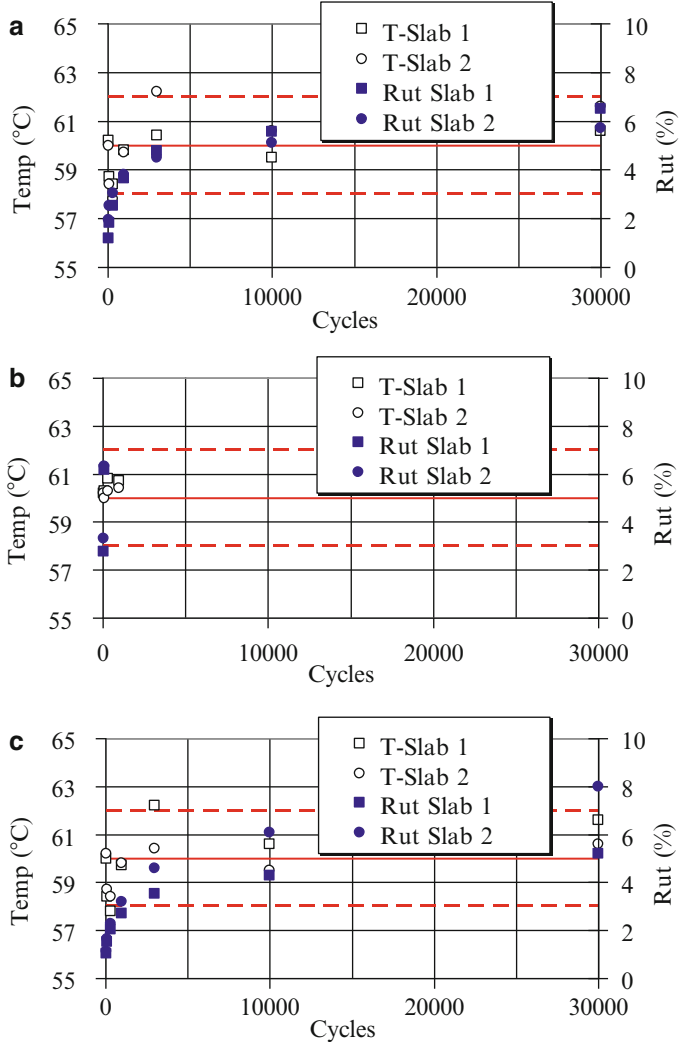


Fig. 4.41 Temperature evolutions under testing of the three bituminous layer systems (A, B and C) tested with the FWTT device (Laboratory I; Regulation: 60°C; lower and upper limits of 58°C and 62°C respectively): (a) System A, (b) System B and (c) System C

The interval of a population mean represents the upper and lower limits whereas the true value is confined (absolute). To establish if two populations could be considered as statistically distinctive, sample means are compared based on the estimated interval of the true value of each population. To do so, we refer to a continuous probability distribution and fix a confidence interval.

Assuming a normal distribution and applying the Student law ($n < 30$), the estimated interval of the population mean for each tested bituminous layer system (X_i^+ , X_i^-), can be calculated by the following equation:

$$X_i = \bar{x}_i \pm \frac{t_{(\alpha/2)}}{\sqrt{n_i - 1}} \bullet s_i \quad (4.11)$$

$$X_i = \bar{x}_i + X_i^+ \quad \text{and} \quad X_i = \bar{x}_i - X_i^-$$

where, n_i refers to the number of measurements for a given tested bituminous layer system, $t_{(\alpha/2)}$ corresponds to the Student t-distribution value for a given confidence interval ($1 - \alpha$) and s_i refers to the standard deviation of the sample.

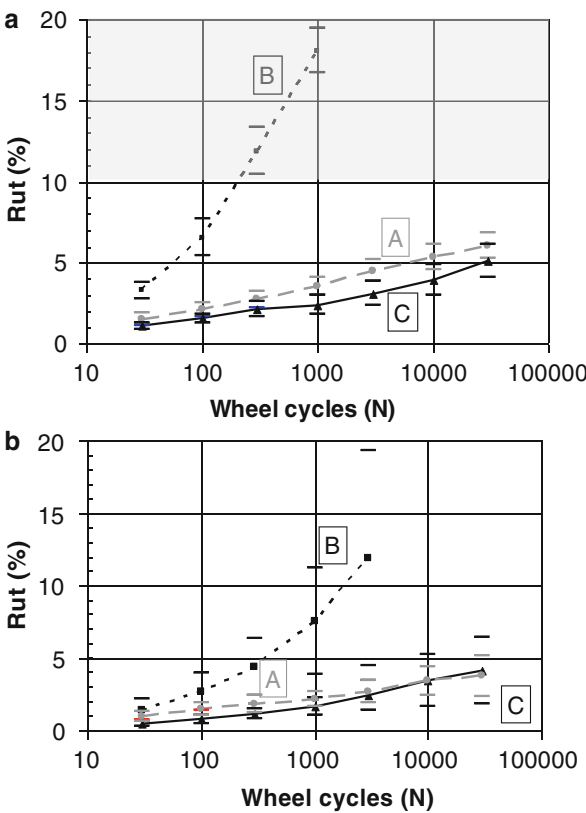
The interval of the population mean, designated by X_i^+ and X_i^- , is a function of the standard deviation of results (s_i) and the number of repetitions (n_i) performed in a series of tests. For a given value of s_i , the interval of the population mean is reduced by increasing the number of repetitions. Nevertheless, for few repetitions, the interval of the population mean could be more spread than the interval of the sample mean ($\bar{x}_i \pm s_i$). Based on the interval of the population mean (X_i^+ , X_i^-), we can statistically compare results of tested systems, and determine whether or not they represent distinctive populations. In Tables 4.4 and 4.5, the last two columns in the tables give the spread of the population mean, calculated according to Eq. (4.11) (X_i^+ , X_i^-). Calculations were done for a confidence interval of 95% ($\alpha = 5\%$), at each measurement step (wheel cycles N of 30; 100; 300; 1,000; 3,000; 10,000 and 30,000).

Performance of Bituminous Layer Systems A, B and C, Given by FWTT

Figure 4.42 shows the relationship between the rut (%) of each tested system (A, B and C) and the number of wheel cycles. The rut values correspond to the sample mean, the average value of the overall test results of a tested system, at each measurement cycle, for a given testing temperature. In Fig. 4.42, for each rut value, the calculated intervals of the population mean are indicated. Figure 4.42a shows results for the 60°C test condition and Fig. 4.42b gives results for the 50°C test condition. For the 60°C test condition, four laboratories were involved and two slabs were tested (repetition: $n_i = 8$). For the 50°C test condition, three laboratories were involved and two slabs were tested. Nevertheless, one set of results was not considered due to erratic performance of the rutting machine. Consequently, results from two laboratories were considered (repetition: $n_i = 4$). For each tested bituminous layer system, we can see that the uncertainty is increased with the number of cycles. For systems A and C, Fig. 4.42 shows a low scattering of results for low rut level. Nevertheless, for system B, where the rut level is higher, the scattering of results is higher at 50°C than at 60°C.

Figure 4.42 shows no differences between bituminous layer systems A and C. Nevertheless, Fig. 4.42 clearly shows that the tested bituminous layer systems

Fig. 4.42 Rut measurements as a function of the number of wheel cycles made with the FWTT: (a) At 60°C and (b) At 50°C



A and C are very different from system B. Results obtained at 50 and 60°C tell a similar story.

Temperature Effect

Figure 4.43 shows the relationship between the rut (%) of each of the tested systems (A, B and C) and the number of wheel cycles using the FWTT for the 60°C test condition (dashed lines) and the 50°C test condition (full lines). For each temperature, the corresponding repetition (n_i) used for calculation is indicated in each graph. As can be noted in Fig. 4.43, results at 50 and 60°C could be considered as two distinct populations for all tested systems.

Rutting Performance According to Standard Requirement

The French specification for this type of bituminous materials requires that the rutting at 30,000 cycles should be lower than 10%. This specification is plotted in

Fig. 4.43 Relationship between the rut, measured with the FWTT at 50 and 60°C, as a function of the number of wheel cycles: (a) System A, (b) System B and (c) System C

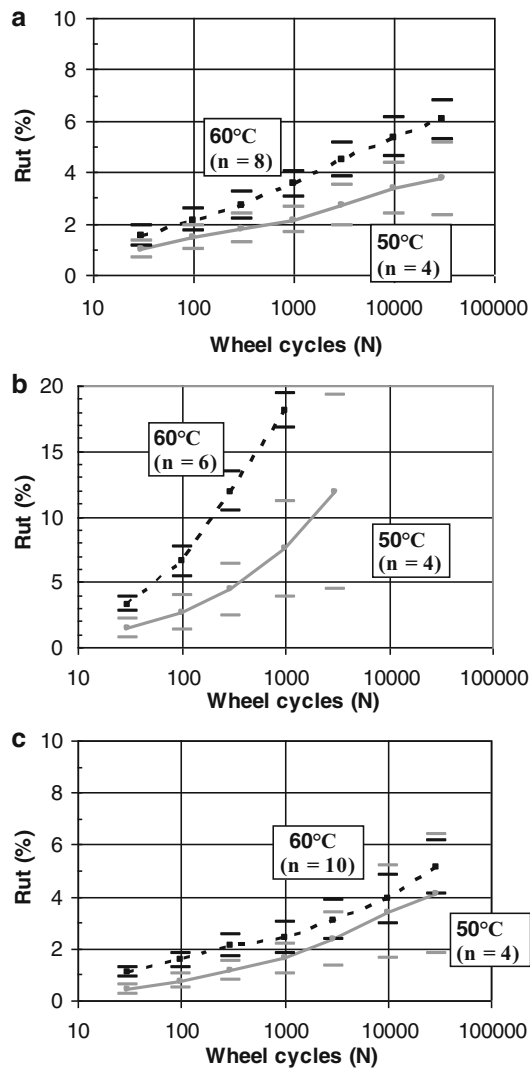


Fig. 4.42a (grey shading). Figure 4.42a shows the rutting performance for the three bituminous layer systems tested with the FWTT at 60°C. As we can see from the laboratory investigation, systems A and C show equivalent performance and can be considered good in terms of the standard requirements. These results are in accordance with observations made on-site. However, laboratory results indicate poor performance of system B and its results do not comply with the specifications of the standard. The rutting resistance of system B is so poor that the test was stopped at about 1,000 wheel cycles. Nevertheless, the rutting performance on-site is good for bituminous layer system B, even after many years of service. So, we can therefore conclude that FWTT and European regulations are severe. On the other hand, wheel

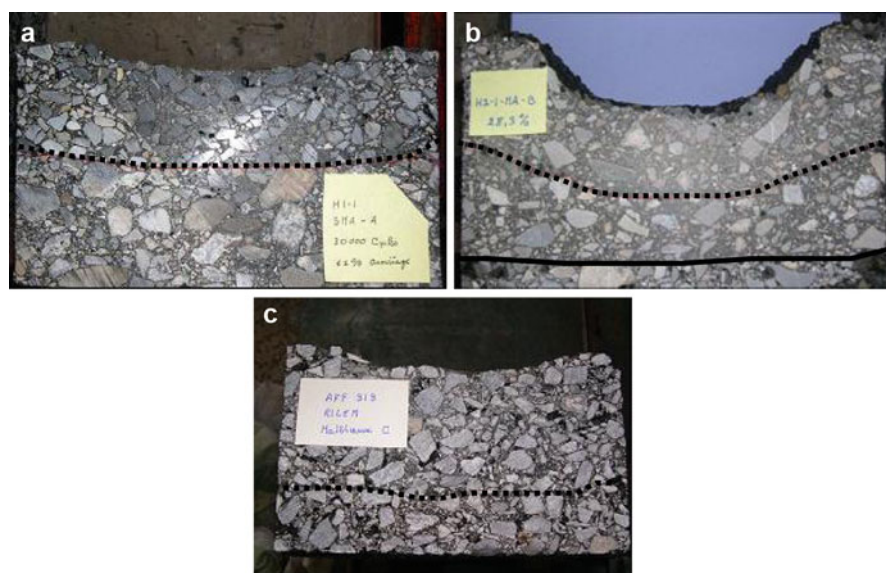


Fig. 4.44 Picture of cut transverse sections for the three bituminous layer systems tested with the FWTT at 60°C: (a) System A, (b) System B and (c) System C

tracking tests are nevertheless a simulation test and results tend to emphasise that it is not appropriate for every kind of bituminous mixtures, such as mastic asphalt (*“Enrobés coulés”*; *“Gussasphalt”*).

Figure 4.44 shows pictures of cut transverse sections of the tested slabs for the three bituminous layer systems. The dashed black lines mark the interface between the surface layer and the bond layer, while the full black lines (system B only) mark the interface between the bond layer and the base layer. As can be seen, the material of the surface and the bond layers of system B moves from under the wheel to the sides of the wheel path.

4.3.1.4 Test Results for the Three Considered Devices (Large and Small)

Two laboratories used other types of WTT (small devices) to evaluate the rut performance of the three bituminous layer systems (Table 4.3). Rutting results for all the considered WTT devices are presented in Fig. 4.45.

In this Figure, it can be seen that the three devices give a different rut evaluation between system B and the two other systems: system B gives much more rutting than the two other systems. In addition, as shown in the rut evolution of system A (Fig. 4.45b), the rutting curves obtained from the three types of device, are different. The two types of small device give a rutting rate that increases at the end of the tests (phase III, see Perraton et al. [106]). For WTT-I and -II devices, the loading conditions seem to be more severe. The stress level for these small devices

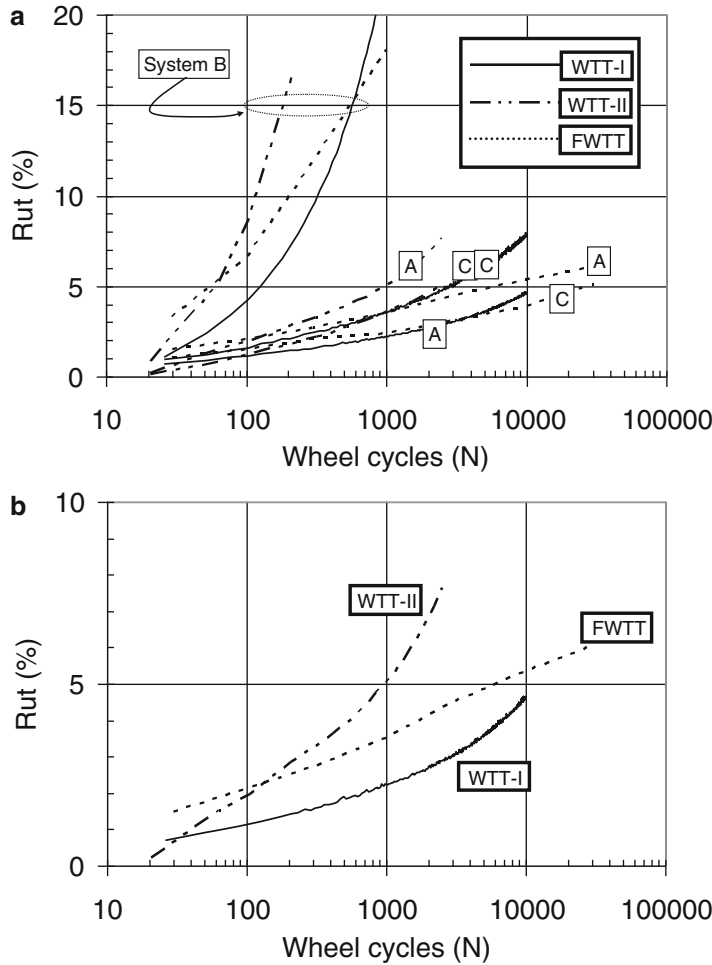


Fig. 4.45 Relationship between the rut (%) and the number of wheel cycles made with small and large devices at 60°C: (a) for the three tested bituminous layer systems and (b) system A only

is higher and the wheel action is more concentrated than for the FWTT. This can create a punching effect at the load location, which could then increase the amount of permanent deformation.

As reported in Table 4.6, in order to compare all the results together, we defined different numbers of wheel cycles according to the WTT devices used to allow the final compilation of rut results. Based on those results, each laboratory established a ranking of the three bituminous layer systems, as reported in Table 4.6 and Fig. 4.46. Figure 4.46 shows that the classification from each laboratory is more or less the same, even for A and C systems, where the rut level is so close that no statistical difference could be detected.

Table 4.6 Overall rut compilation for the WTT and the FWTT devices and final ranking (1, 2 or 3) from each laboratory for each tested system

		System					
		A		B		C	
		SMA 11S		GA 11T		BBSG	
Device	Lab	Ranking	Rut (%)	Ranking	Rut (%)	Ranking	Rut (%)
FWTT	I	1	6.1 ^a	3	10.9 ^b	2	6.6 ^a
	III	1	5.7 ^a	3	14.1 ^b	2	6.4 ^a
	IV	1	4.0 ^a	3	12.3 ^b	2	5.9 ^a
	V	2	6.5 ^a	3	10.1 ^b	1	4.5 ^a
WTT	VI	1	2.9 ^c	3	7.9 ^d	2	4.8 ^c
	VII	2	7.6 ^c	3	16.5 ^d	1	5.1 ^c

^a Values after 30,000 wheel cycles

^b Values after 300 wheel cycles

^c Values after 2,520 wheel cycles

^d Values after 220 wheel cycles

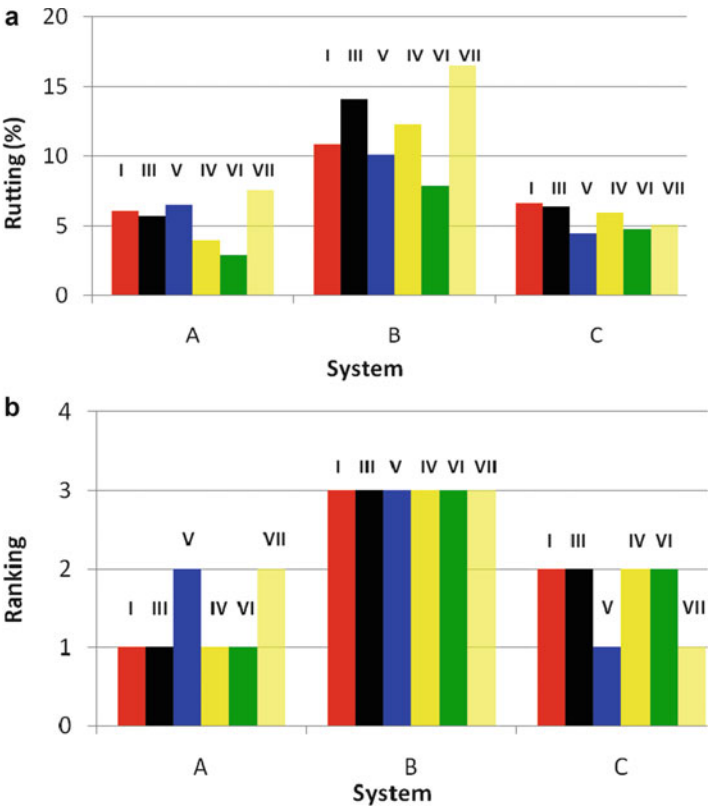


Fig. 4.46 Final compilation of rutting results and ranking established from each participant laboratory (60°C): (a) Rutting (%) and (b) ranking



Fig. 4.47 Picture showing the surface course of the road after 10 years in service for bituminous layer system B

4.3.1.5 Onsite Performance and Comparison with Laboratory Performance

On Road Performance for Systems A and B

The pavements with bituminous layer systems A and B were constructed in 1998 and 1995 respectively. Samples were cut onsite just after construction and stored in a laboratory room at ambient temperature (22°C) until the rutting tests in 2007. The delay between the sampling and the testing time may have had an influence on the rutting performance of materials. Nevertheless, it could be assumed that the storage period would improve the rutting performance and could not be considered as a negative effect on the rutting performance of systems A and B in laboratory.

In 2008, a visual evaluation of the rutting shows a very good performance on the road for these two bituminous layer systems. Moreover, no apparent difference could be noted between system A and system B. Presently, the rutting level is low and very acceptable for both systems. Figure 4.47 shows a picture of the quality of the road surface of bituminous layer system B after 10 years in service.

Performance of System C on the Circular Test Track and Comparison with Laboratory Measurements

Bituminous layer system C was cut from the LCPC test track in Nantes (France) in 2005 and its construction and rutting testing on the test track were carried out in 2002. For the on-site measurement, no measurement was done to differentiate rutting within the bituminous layers itself and that within the granular materials

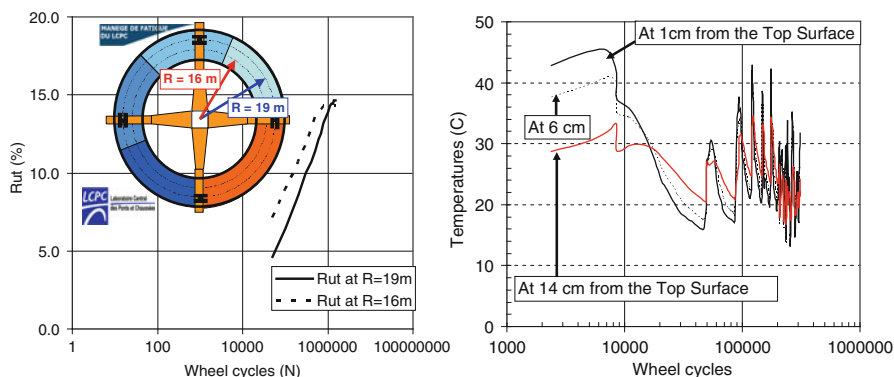


Fig. 4.48 Relationship between rut depth and number of wheel cycles, measured on the test track in Nantes for bituminous layer system C (Load : 65 kN): (a) Rut versus wheel cycles and (b) Temperature versus wheel cycles

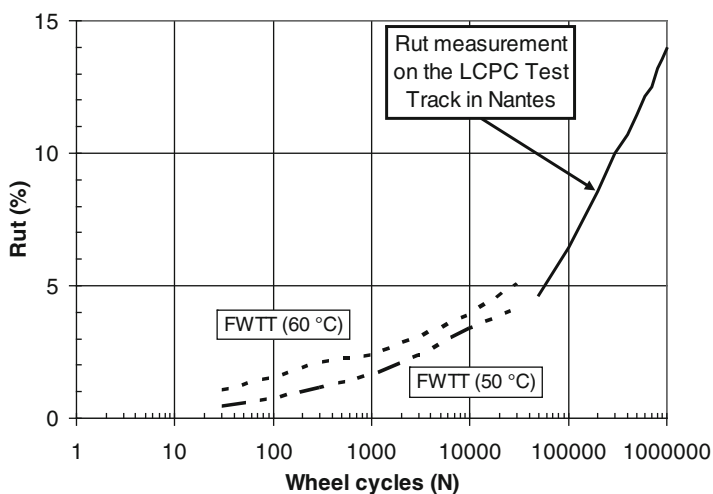


Fig. 4.49 Relationship between the rut depth and the number of wheel cycles measured on the LCPC Test Track ($R = 19$ m) and on the FWTT in laboratory

and subgrade. Nevertheless, the total deflection was expressed as a function of the full thickness of bituminous layers of the pavement only.

The rut evolution of bituminous layer system C measured on the test track is presented in Fig. 4.48a. Results show a good performance on-site. Fig. 4.48b shows that the mean temperature during testing on the test track was between 16 and 34°C at a depth of 14 cm from the top surface of the pavement.

For bituminous layer system C, the rut measurement on pavement was conducted on the LCPC test track and could be compared with the laboratory investigation. Figure 4.49 shows the relationship between the rut depth and the

number of wheel cycles measured on the LCPC test track and in the FWTT at 50 and 60°C.

The first measurement carried out on the LCPC Test Track was at 50,000 cycles, so we cannot directly compare the shape of the curves together. Nevertheless, results at 60°C with the FWTT seem to correlate well with the measurements carried out on the LCPC Test Track. As noted in Fig. 4.48b, the top surface temperature on-site was not close to that used in the laboratory test.

4.3.1.6 Conclusion on Wheel Tracking Test Campaign “A”

Three different bituminous layer systems (A, B and C) were compared to evaluate the abilities of “small” and “large” WTT devices to characterise rutting performance of road materials. A series of tests were performed at 50 and 60°C. Based on results obtained, the following conclusions can be made when considering all the devices:

1. Small and large WTT devices give similar ranking for the three bituminous layer systems tested;
2. The rutting rate for “small” WTT devices is faster than for “large” WTT devices. Rutting curves from the two classes of WTT (“large” (French WTT) and “small”) are noticeably different. Small devices seem more aggressive and an increase of rutting rate is observed at the end of the tests.

Five countries used the French Wheel Tracking Tester (FWTT) device. FWTT results show that:

1. Systems A and C have equivalent performance and can be considered as good in terms of the requirements of the European Standard. However, results of system B indicates poor performance and do not comply with the specification of the standard;
2. Results at 50 and 60°C can be statistically considered as two distinct populations for all tested systems (A, B and C);
3. The trends observed in rutting performance at 50 and 60°C are similar.

Finally, even though the rutting performance of system B measured in laboratory on FWTT devices was out of specification, the rutting performance on the road is still good after 10 years in service. It can be concluded that FWTT and European Standards are both severe and conservative.

Table 4.7 Mix design of the asphalt concrete tested in this study. Binder content is expressed in terms of external percentage

Origin quarry	Class [mm]	Mass ratio [%]	Bulk density
Noubleau	10/14	23.5	2.889
Noubleau	6/10	22	2.829
Noubleau	4/6	10	2.833
Noubleau	2/4	11	2.863
Cusset	0/2	32	2.690
Airvault	filler	1.5	2.706
Total Agg.		100	
Binder (50/70PmB EVA)		5.55 ^a	
Total		105.55	

^a Part by weight of aggregate

4.3.2 Campaign “B” on Mixtures Made with Polymer Modified Bitumen (AC-PmB) and Using French Wheel Tracking Test (FWTT)

4.3.2.1 Introduction

The RILEM ATB testing program campaign B, on mixtures made with polymer modified bitumen (AC-PmB) was planned in order to check the applicability of the French wheel tracking test device (FWTT) for such modified materials. As only one type of device and one type of materials was selected the campaign can be considered as a round robin test.

Eight laboratories took part in the Campaign, namely, Belgium Road Research Centre (Belgium), University of Lyon/ENTPE (France), Eiffage Travaux Publics (France), Road and Bridge Research Institute (IBIDIM, Poland), Laboratoire Central des Ponts et Chaussées (France), University of Quebec/LUCREB (Canada), University of Nottingham/NTEC (United Kingdom), TOTAL (France).

4.3.2.2 Materials, Specimens and Tests Performed

Material Design

The asphalt concrete (AC) tested in this study was made with a plastomer type polymer modified binder. The constituents are presented in Table 4.7 and the grading curve is given in Fig. 4.50.

Coarse Aggregates have been extracted from the Noubleau quarry (France), the fine aggregates (particles less than 2 mm) and the filler came from the Cusset quarry (France) and the Airvault quarry (France) respectively. The mineralogical nature of Noubleau, Cusset and Airvault quarries are diorite (igneous rock), andesitic tuff

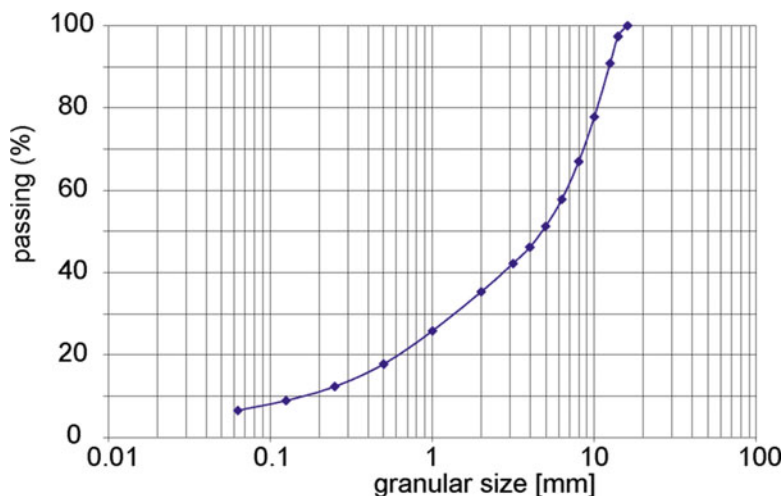


Fig. 4.50 Grading curve of the aggregates of the mix design of this study

(igneous rock) and limestone (sedimentary rock), respectively. The sand (0/2 mm) from Cusset quarry has been milled by means of a rod mill. This sand is more rounded than typical crushed sands, which means that the mixture is more sensitive to rutting (LCPC [93]). The binder is a polymer modified bitumen. The polymer added in the bitumen is an Ethyl-Vinyl-Acetate (EVA), which is plastomeric in nature. The considered mixture can be classified as a BBSG (Béton Bitumineux Semi-Grenu).

Specimen Manufacturing

Slabs Manufacturing

AC slabs have been manufactured in the laboratory according to the standard EN 12697–33 using the French roller compactor according to a “strong” compaction schedule. All the slabs have been prepared in the same laboratory (LCPC Nantes), in order to avoid problems of reproducibility in manufacturing. However, for practical reasons and in order to limit the quantity of slabs to manufacture, it has been decided to prepare big slabs $600 \times 400 \times 100$ mm, that were then cut into two specimens to be tested with the FWTT device ($490 \times 175 \times 100$ mm) (see Fig. 4.51).

Remarks on the Compaction Phase

During the compaction phase using the French Roller Compactor, slabs have shown a “mattress effect” (bumps can be observed around the wheel). Samples behave like fine saturated soils. This behaviour during compaction is usually observed when the

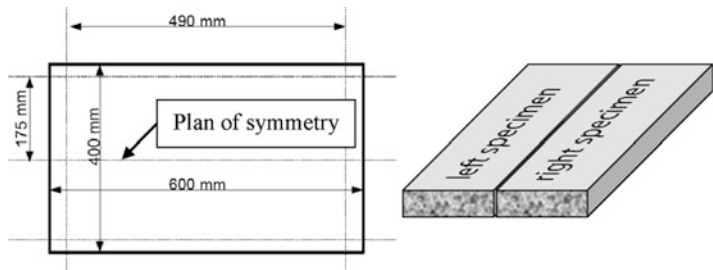


Fig. 4.51 A big manufactured slab gives two specimens tested with the FWTT device

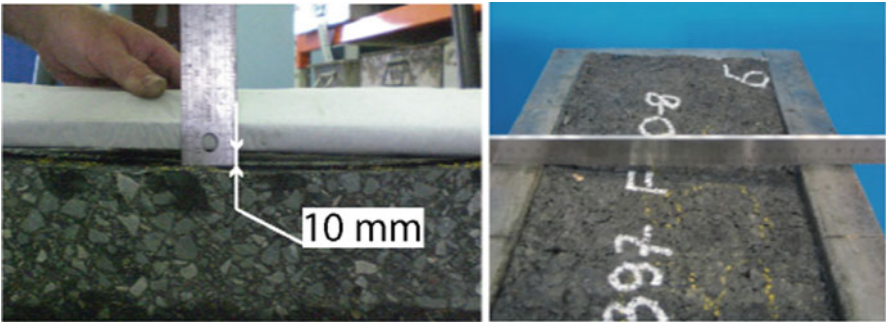


Fig. 4.52 Problems observed during slabs compaction phase: for a size of $490 \times 175 \times 100$ mm, a settlement of 10 mm in the middle of the slab can be observed for some slabs

Table 4.8 Theoretical void content and measured void content of an AC slab

	Target	Specimen 1	Specimen 2	Mean value
Void content	4–8%	2.3%	3.02%	2.6%

bituminous mixture has reached a high level of compaction. As a result, slabs have exhibited a curved top surface instead of the expected flat surface (see Fig. 4.52). Over a length of 490 mm, the curvature depth could reach as much as 10 mm. This was also found to occur more in the middle of the slab than at the edges.

Density Analysis

A measurement of void content (void volume in percent of total volume) of two specimens coming from one slab has been performed by one laboratory. Gamma rays have been used to assess the void content, according to the standard (EN 12697–7 [78]). Results are presented in Table 4.8.

According to the standard EN 13108–1 [79], annex E, porosity for such a material is expected between 4 and 8%, for a good behaviour on site. The obtained

mean value shows that our material is over compacted, which makes it more sensitive to the rutting phenomena.

4.3.2.3 Anti-overheating and Anti-sticking Procedures

Some laboratories have encountered problems of overheating of specimens and sticking between tyre and specimen during previous rutting tests (Brosseau and Hieunaux [18]). These two phenomena could create a maximum temperature exceeding the standard recommendation. In addition, these phenomena, whose influence is different at different laboratories, are factors for discrepancy. Even though from discussions within the group it appeared that sticking problems were noticed by only a few numbers of the participants, two specific procedures, an anti-sticking and an anti-overheating procedure have been proposed in order to mitigate against these problems. The Round Robin Test was organized to assess the potential of these procedures to prevent these phenomena and to limit the scattering of the results among the different laboratories. The objective is to improve the reproducibility of the test method. These procedures are described in the following sections.

Anti-overheating Procedure

For FWT tests on a classical material, no overheating is usually observed. The following procedure has been specifically developed in this Round Robin Test to prevent overheating.

An FWTT device with four temperature probes is considered: two probes enable respectively to measure the temperature ($T^{\circ}\text{C}_{\text{specimen1}}$) of specimen 1 (right) and temperature ($T^{\circ}\text{C}_{\text{specimen2}}$) of specimen 2 (left). These probes are installed in the middle of the slabs, just beside the wheel path, at the inner side of the machine, which are the places where the highest temperatures are normally expected. Another probe put inside the specimen 1 is used to regulate the temperature of the thermal chamber and another probe enables to measure the air temperature inside the thermal chamber (see Fig. 4.53). For a test performed at 60°C (resp. 50°C), this procedure aims to keep the temperatures measured inside the samples between 58°C (resp. 48°C) and 62°C (resp. 52°C) throughout the test. This is described in Table 4.9.

Anti-sticking Procedure

This procedure has mainly been developed at LCPC by Vincent Dubois, after observing sticking problems during previous tests on AC-PmB. It consists of lubricating the tyre with a mixture of Glycerised Sodium Oleate (GSO). GSO is a highly greasy soap. In addition, a “sulfurised paper” (cooking paper) is laid on the specimens and also pasted with GSO. Each time the door of the apparatus is open,

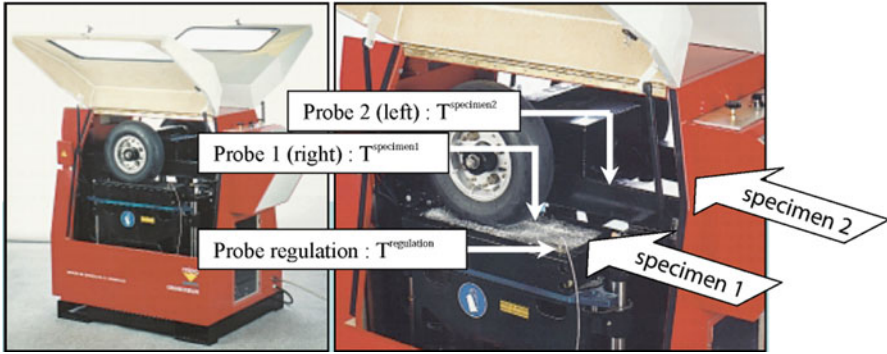


Fig. 4.53 French Wheel Tracking Test. Position of probes

Table 4.9 Anti-overheating procedure for tests respectively performed at 60°C (respectively 50°C)

	Test launching
Condition 1	If $T^{\circ}C^{\text{specimen}1}$ or $T^{\circ}C^{\text{specimen}2}$ reaches 62°C (resp. 52°C) while increasing:
Step 1	Loading and thermal chamber regulation (heating) are both stopped. FWTT device doors are opened
Step 1 bis	If anti-sticking procedure, spread GSO (Glycerized Sodium Oleate) and/or place new paper if necessary
Condition 2	When $T^{\circ}C^{\text{specimen}1}$ or $T^{\circ}C^{\text{specimen}2}$ reaches 58.5°C (resp. 48.5°C) while decreasing,
Step 2	The doors of the FWTT are closed, the thermal chamber regulation (heating) is restarted, the regulation temperature for the chamber is fixed at 58.5°C (resp. 48.5°C)
Condition 3	After 45 min of regulation (to let specimens reach thermal equilibrium before reloading it!)
Step 3	Cyclic loading is restarted at a regulation temperature of ($T^{\circ}C^{\text{regulation}}$) 58.5°C (resp. 48.5°C)

the paper is checked. If damaged it is changed and GSO is spread on it and on the tyres (Fig. 4.54). The apparatus can be opened either in order to carry out measurement or to follow the anti-overheating procedure.

4.3.2.4 Tests Performed During Campaign B

All the tests have been performed following the anti-overheating procedure, as closely as possible. Eight laboratories, namely, Belgium Road Research Center (Belgium), University of Lyon/ENTPE (France), Eiffage Travaux Publics (France), Road and Bridge Research Institute (IBIDIM, Poland), Laboratoire Central des Ponts et Chaussées (France), University of Quebec/LUCREB (Canada), University of Nottingham/NTEC (United Kingdom), TOTAL (France), participated



Fig. 4.54 Anti-sticking procedure. It consists of placing a sulfurized paper on the specimen and spreading a greasy soap on it and on the tyres

Table 4.10 Tests dispatched between the different laboratories: the figure indicates the number of tested specimens; 1 test needs 2 specimens (1 slab). AS0 means “without anti-sticking” and AS1 means “with anti-sticking”

Test/Lab	1	2	3	4	5	6	7	8
50°C AS0	2	2		2				
50°C AS1		2				2		
60°C AS0		2	2	2	2 ^a + 2 ^a + 2 ^a		2	
60°C AS1	2 + 2 ^b		2 + 2	2		2	2	
Rheological tests								2

^a Tests performed with two different tyres, one side of machine fitted with a tyre from Lab5 and the other a tyre from Lab2

^b The test is performed with two different tyres (one from Lab1 and one from Lab2)

in the campaign. Seven laboratories made FWTT and one laboratory characterised the rheological properties of the tested mixture (AC-PmB).

In the next sections, laboratories are identified by numbers in random order to keep them anonymous. It was decided to perform tests both at 50 and 60°C. Table 4.10 indicates the type and number of tests performed in the different laboratories. One FWTT needs two specimens (left and right on the device), and therefore one “big” slab from the French wheel plate compactor.

4.3.2.5 Rheological Tests Results (Complex Modulus Tests)

Tension-compression complex modulus tests have been performed by University of Lyon/ENTPE in order to quantify viscoelastic characteristics of the tested PmB material. The test equipment used to apply sinusoidal loading in tension and compression (through zero) on cylindrical specimen of asphalt mixture at different frequencies and different temperatures, is presented in Fig. 4.55. This tension-compression (push-pull) test creates a homogenous stress and strain field in the middle part of the tested specimen (Di Benedetto et al. [59, 60]).

The specimen is loaded at seven different frequencies (*f*) from 0.01 to 10 Hz and eight different temperatures (*T*) from −8.4°C to 49.9°C. For each pairing of temperature and frequency (*T* – *f*), three levels of axial strain amplitude

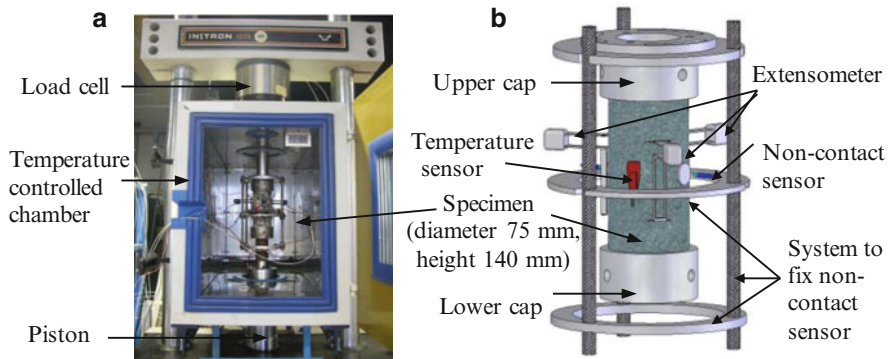


Fig. 4.55 (a) General view of the test equipment, (b) closer view of the specimen showing device to measure axial strain, radial strain and temperature

(<125 $\mu\text{m/m}$) were applied. The results presented are obtained by linear fitting of the measured data and extrapolation at the strain amplitude equal to 50 $\mu\text{m/m}$.

Sinusoidal axial strain (ε_1) (average of three extensometers) is applied, sinusoidal axial stress (σ_1) is measured from load cell and sinusoidal radial strain (ε_2) is deduced from two radial non-contact sensors (Fig. 4.55b):

$$\varepsilon_1(t) = \varepsilon_{01} \sin(\omega t) \quad (4.12)$$

$$\sigma_1(t) = \sigma_{01} \sin(\omega t + \phi_E) \quad (4.13)$$

$$\varepsilon_2(t) = -\varepsilon_{02} \sin(\omega t + \phi_v) \quad (4.14)$$

where ϕ_E is the phase angle between the axial strain and the axial stress, and ϕ_v is the phase angle between the axial strain and the opposite of radial strain. ε_{01} , ε_{02} and σ_{01} are respectively the axial strain amplitude, the radial strain amplitude and the axial stress amplitude.

Complex modulus E^* and complex Poisson's ratio v^* are obtained considering Eqs. (4.15) and (4.16), where $|E^*|$ and $|v^*|$ are respectively their norms and j is the complex number defined by $j^2 = -1$:

$$E^* = \frac{\sigma_{01}}{\varepsilon_{01}} e^{j\phi_E} = |E^*| e^{j\phi_E} \quad (4.15)$$

$$v^* = \frac{\varepsilon_{02}}{\varepsilon_{01}} e^{j\phi_v} = |v^*| e^{j\phi_v} \quad (4.16)$$

Test results are presented in Fig. 4.56, for both complex Young's modulus (E^*) and complex Poisson's ratio (v^*). The three plotted master curves use the same shift factor (a_T) (Fig. 4.57), which is the first significant result. This figure also highlights that such PmB mix does not follow the Time-Temperature Superposition Principle (TTSP), as a unique curve is not obtained in the Black representation (Fig. 4.56d).

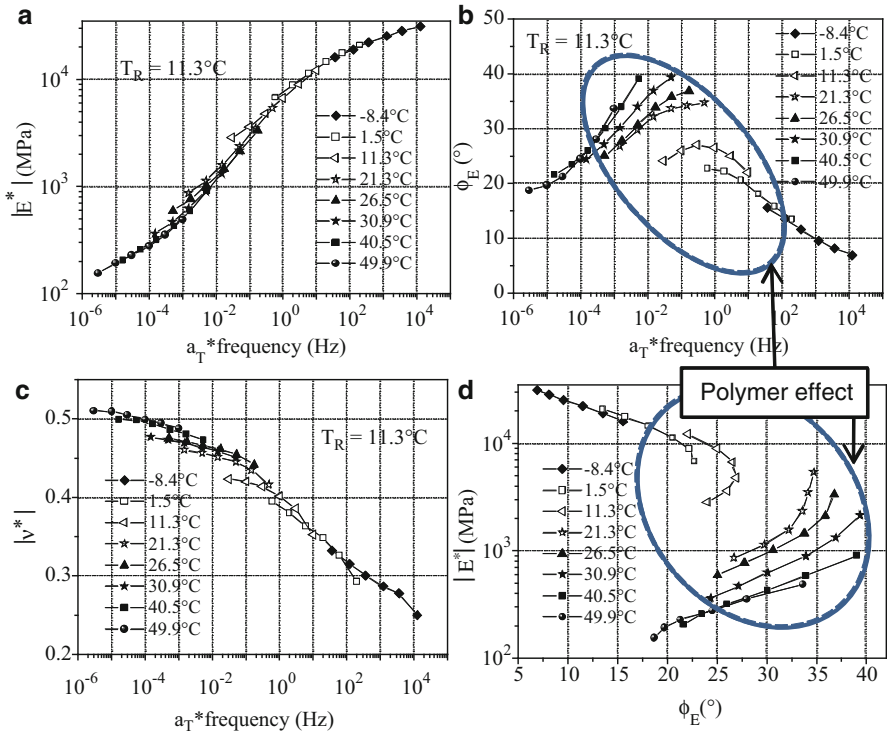


Fig. 4.56 Results of complex modulus tests on the PmB mixture; master curves of: norm of complex modulus (a), phase angle of complex modulus (b), norm of complex Poisson's ratio (c) and Black diagram of complex modulus (d)

Meanwhile the so called “Partial Time-Temperature Superposition Principle” (PTTSP), as defined by Olard and Di Benedetto [103] can be considered because the shifting procedure gives a rather unique master curve only for the norm of the modulus (and not the phase angle). It can also be observed Fig. 4.56 that the PTTSP can be considered for the Poisson's ratio.

It is well known that the rutting process is much more significant at high temperatures (above 40°C). For this range of temperatures EVA polymer has an important influence as can be deduced from the shape of the Black curve. This influence is mainly characterised by, (i) a decrease of phase angle with temperature and frequency, and (ii) the influence of the temperature on the shape of the curve when changing frequency (TTSP not respected).

As clearly shown in this chapter, temperature increases during the wheel tracking test (WTT), which is a strong biased effect of the test. This increase is due to viscous dissipated energy that is transformed into heat (Di Benedetto et al. [68]). In order to evaluate this increase, Fig. 4.58 gives the values of the viscous dissipated energy per cycle and unit volume (W_N) according to Eq. (4.17) for each considered experimental frequency and temperature. The two plotted master curves in Fig. 4.58

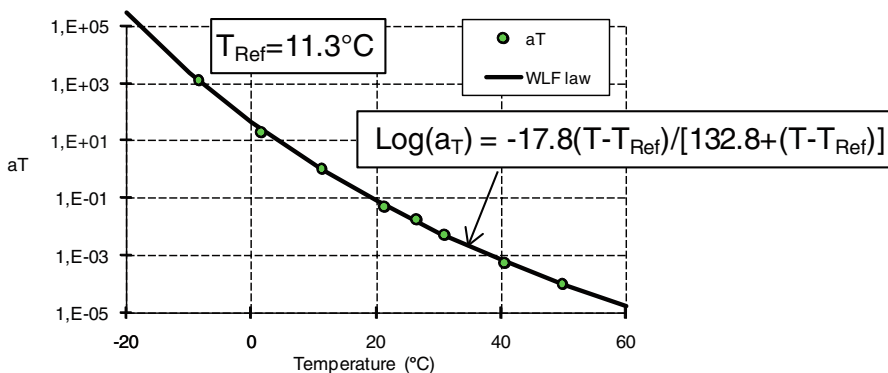


Fig. 4.57 Shift factor a_T for complex modulus and Poisson's ratio of tested bituminous mix (Fig. 4.56) at a reference temperature $T_{ref} = 11.3^\circ\text{C}$: experimental results and WLF law fitting

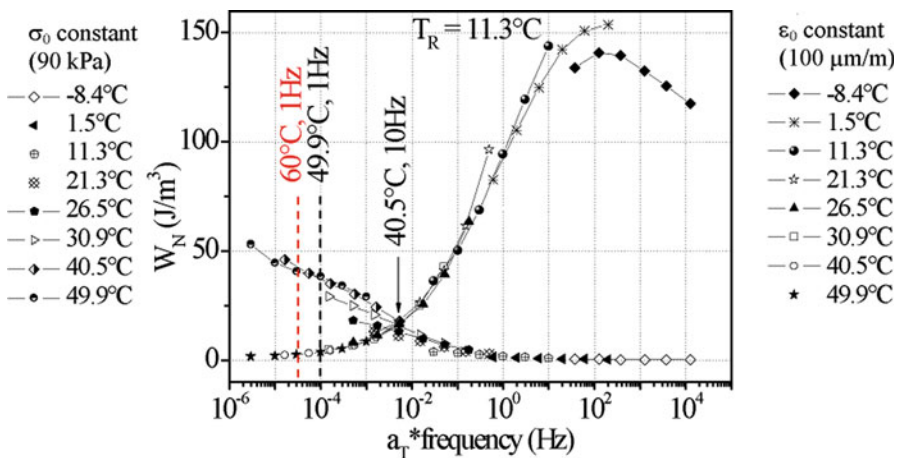


Fig. 4.58 Master curves of viscous dissipated energy per cycle W_N at a reference temperature ($T_R = 11.3^\circ\text{C}$) for two cases: strain control tests ($\varepsilon_0 = 100 \mu\text{m/m}$) and stress control tests ($\sigma_0 = 90 \text{ kPa}$). Tests are the same as Fig. 4.52

represent results for cycles at a constant strain amplitude ($\varepsilon = 100 \mu\text{m/m}$) and cycles at a constant stress amplitude ($\sigma = 90 \text{ kPa}$). The shift factor used for master curves of Fig. 4.56 is considered.

$$W_N = \pi \sigma \varepsilon \sin(\varphi) \quad (4.17)$$

W_N is the dissipated energy per cycle and unit volume at cycle N , where the strain amplitude, stress amplitude and phase angle are ε , σ and φ respectively.

Figure 4.58 reveals that TTSP is valid for W_N . In this figure the two values of (50°C , 1 Hz) and (60°C , 1 Hz) are indicated. These two values correspond to the

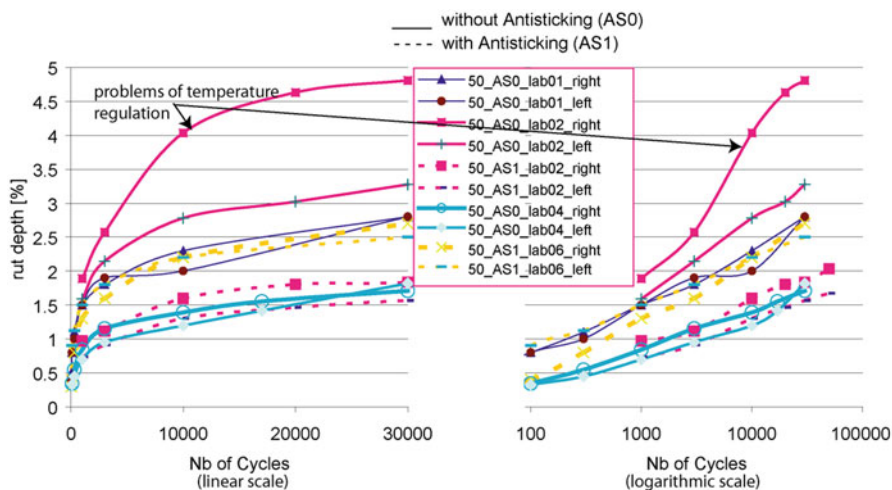


Fig. 4.59 Rutting tests performed at 50°C, *right & left* rut depth values. Both linear and logarithmic scales are presented

temperatures and frequency considered during the WTT campaign. As the WTT is a stress controlled test (a constant load is applied to the wheel that creates an average pressure of 600 kPa at the surface), emphasis should be given to stress control master curve. Dissipated energy at 50 and 60°C for stress control tests at 1 Hz are rather high, which explains why heating is so important for these kinds of materials.

The results presented in this chapter are considered for the FEM calculation of FWTT performed by a sub-team of RILEM TC 206-ATB TG3, which is presented in the next section.

4.3.2.6 Results for Rutting Tests at 50°C

Results of RRT performed with the FWTT at 50°C are presented in Fig. 4.59 (left and right specimens), with the rut depth in percent plotted as a function of the number of cycles. Rut depth is plotted as a function of the number of cycles on both a linear and a logarithmic scale. Full lines represent tests performed without the anti-sticking procedure (AS0), whereas dashed lines correspond to tests performed with the anti-sticking procedure (AS1). Each result is the average value of rut depths measured on left and right specimens (see Fig. 4.53).

The left and right results are quite similar for each laboratory, except for lab2 (results AS0-lab2). A failure of the temperature probe was noticed during this test, which is perhaps responsible for the higher level of rutting observed on the right specimen. It is possible that the right specimen has been submitted to increased overheating, but heterogeneity of the test specimen could also explain the difference. In the following analysis, this result (AS0-lab2-right) is not taken into

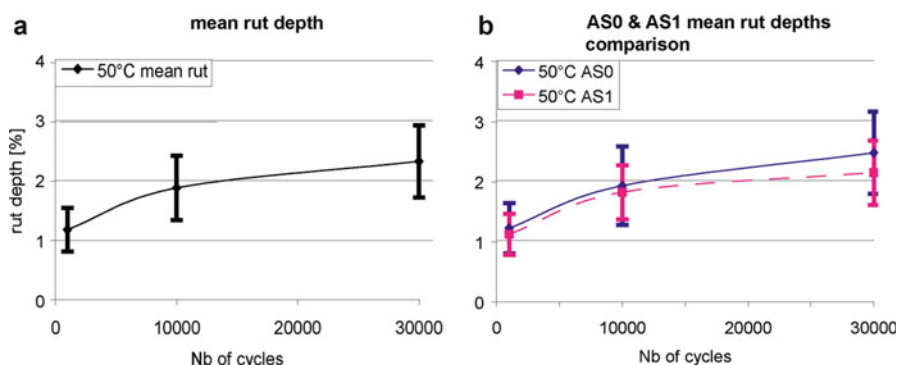


Fig. 4.60 (a) Mean rut depth values for all tests performed at 50°C. (b) Comparison of AS0 and AS1 results at 50°C, in terms of mean values and standard deviations. All tests of Fig. 4.59 are considered except AS0-lab2-right

account. In Fig. 4.60a, the mean rut depth and standard deviation of all tests performed at 50°C is presented. In Fig. 4.60b, a comparison of mean AS0 and mean AS1 results is shown. At 30,000 cycles, rut depths vary between 1.5 and 3.3%. AS1 results (dashed lines) seem to be lower (13% maximum of difference) and less scattered (20% maximum of difference) than the AS0 (full lines) procedure. However, according to the number of results this difference cannot be considered to be significant. The comparison between AS0 and AS1 results will be discussed in Sect. 4.3.2.8.

4.3.2.7 Results for Rutting Tests at 60°C

Results of RRT performed with the FWTT at 60°C are presented in Figs. 4.62 and 4.63. The rut depth (in percent) is a function of loading cycles. Both linear and logarithmic scales are plotted for the number of loading cycles. Full lines represent tests performed without the anti-sticking procedure (AS0), whereas dashed lines represent tests performed with the anti-sticking procedures (AS1). In Fig. 4.62, the rut depth measured for each specimen, left and right, is plotted. In Fig. 4.63, each result is a mean value of rut depths measured under the left and right tyres.

One specimen, AS1lab6 right, showed very significant rutting, with upheaval observed at the edge of the wheel track. According to the observed curved top surface of the slab, this specimen was probably over-compacted before test. It has been deleted for the presented analysis (see Fig. 4.61).

Lab1 AS1, lab3 AS1, lab4 AS1, lab5 (2 and 3) AS0, lab6 AS1 and lab7 AS1, give similar results for the left and right specimens. At 30,000 cycles, an important difference is observed for lab4 and lab6 for the procedure without antisticking (AS0 results) between left and right results. However, in most of cases, differences do not exceed 2% of rut depth at 30,000 cycles.

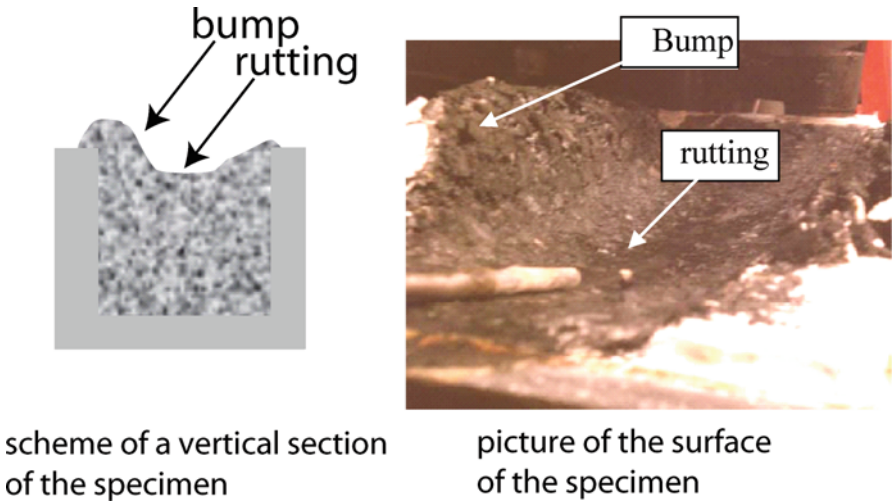


Fig. 4.61 An important bump can be observed for the “60_AS1_lab6_right” result

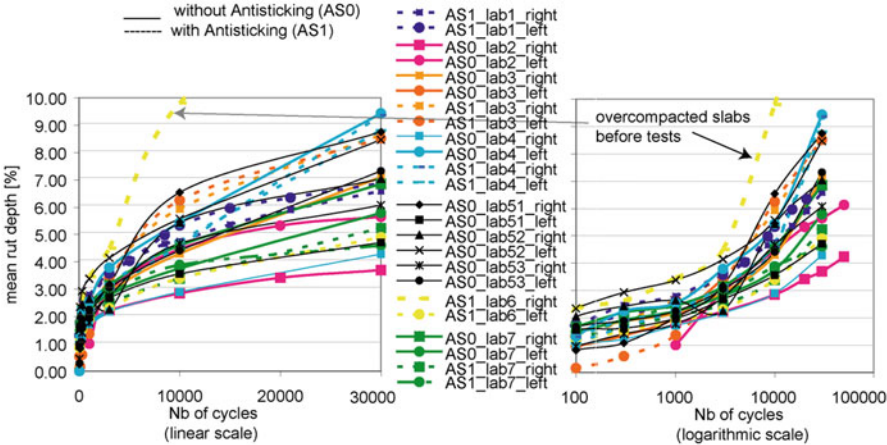


Fig. 4.62 Rutting tests performed at 60°C, *right & left* rut depth values. Both linear and logarithmic scales are presented

The reproducibility R of this RILEM Round Robin Test has been assessed. It is defined as:

$$R = 1.96\sqrt{2\sigma_R^2} \tag{4.18}$$

where σ_R is the standard deviation of all the tests performed. Results are presented in Table 4.11. The reproducibility values “ R ” of this RILEM Round Robin Test are higher than those given by the standard, which indicates less reproducibility in this

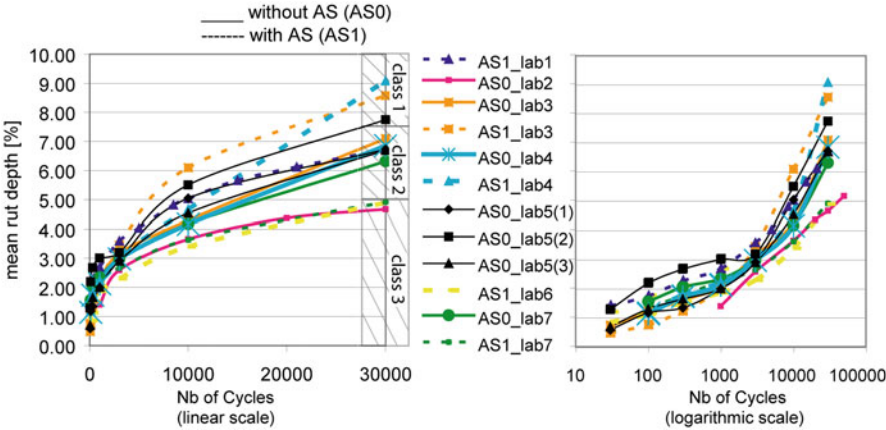


Fig. 4.63 Rutting tests performed at 60°C, mean rut depth values. Performance class considering French design method is also indicated

Table 4.11 Comparison of AS0 and AS1 results at 60°C, in terms of mean values, standard deviations and variances. The t-test cannot show any significant difference between the mean values of AS0 and AS1 results

Nb of cycles	1,000	3,000	10,000	30,000
Mean rut depth [%] AS0	2.14	2.97	4.50	6.55
variance AS0	0.30	0.32	1.16	3.03
Standard deviation AS0	0.55	0.57	1.07	1.74
Mean rut depth [%] AS1	2.26	3.10	4.70	7.06
variance AS1	0.20	0.24	1.05	3.40
Standard deviation AS1	0.45	0.49	1.03	1.85
Mean rut depth [%]	2.19	3.02	4.58	6.76
Standard deviation	0.50	0.53	1.03	1.76
Reproducibility R	1.40	1.48	2.90	4.92
Standard reproducibility (EN 12697-22)	0.97	1.32	1.20	1.16
Last RRT reproducibility (Poirier 2010)			3.88	4.59
t-test : $\text{mean}_{\text{AS0}} = \text{mean}_{\text{AS1}}$	0.58	0.54	0.67	0.52
p-value				

case. However, it has to be remembered that the values given in the standard are based on results of a Round Robin Test performed in 1992, involving 12 laboratories, using specimens of a typical French asphalt concrete, manufactured by a single laboratory. The results are quantitatively more comparable to those obtained during the last French Round Robin Test, performed in 2010, involving 33 laboratories, also using specimens of a typical French asphalt concrete, manufactured by a single laboratory (see Table 4.11). This second comparison seems to be more appropriate, because tests are quite recent. According to these results, it can be concluded that test performed on PmB-AC are of the same order of reproducibility than those performed on typical asphalt concrete without polymers.

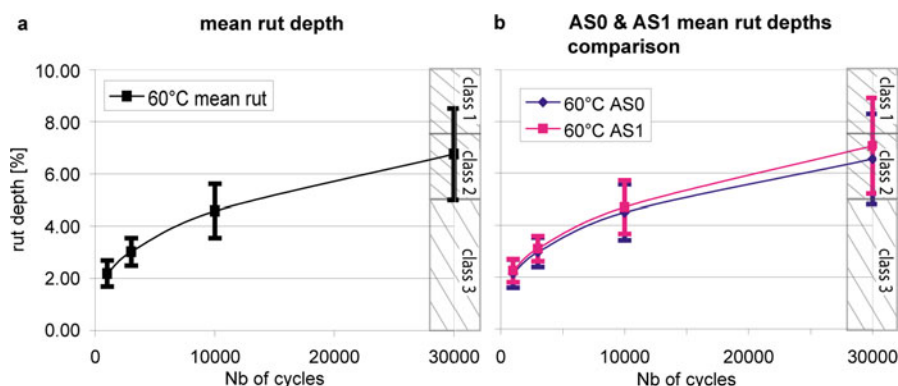


Fig. 4.64 (a) Mean rut depth values for all tests performed at 60°C. (b) Comparison of AS0 and AS1 results at 60°C, in terms of mean values and standard deviations. All tests of Fig. 4.62 are considered except AS1-lab6-right. Performance class considering French design method is also indicated

As the modulus at 15°C and 10 Hz is higher than 7,000 MPa (Fig. 4.56), the material could be classified according to French standard NF EN 13108–1 [79] as a BBSG of class 2 or 3. WTT results at 60°C presented in Figs. 4.63, 4.64 and 4.69, indicate that it should be ranked either in class 1, 2 or 3, following the laboratory and the type of tyre.

The comparison of AS0 and AS1 results will be discussed in the following section.

4.3.2.8 Influence of the Anti-sticking Procedure

At 50°C, results with and without the anti-sticking procedure are compared in Fig. 4.60. Lab2 has performed tests both with and without the anti-sticking procedure. According to these results only, the anti-sticking procedure seems to slightly decrease the level of rutting. However, according to the results of all participants, AS0 and AS1 results are of the same order of scattering and mean values. The conclusion is therefore that no significant influence of the anti-sticking procedure can be observed.

At 60°C, results with and without anti-sticking procedure are compared in Fig. 4.64b. Lab3, Lab4 and Lab7 have performed tests both with and without the anti-sticking procedure. According to results presented in Figs. 4.62, 4.63 and 4.64, the anti-sticking procedure tends to increase the level of rutting for lab4, which is not an expected result. For lab7, this procedure seems to decrease the level of rutting. For lab3, one of the two specimens tested at 60°C without the procedure showed sticking with the tyre and reached such a high level of rutting that it was impossible to measure. Thus, no trend can be seen considering the influence of the

anti-sticking procedure on results lab by lab. It can however be noticed that without anti-sticking procedure, sticking can be observed.

A comparison between the mean AS0 rut depths and the mean AS1 rut depths is presented in Fig. 4.64. Standard deviations intervals are also plotted in this figure. It reveals that the average values of AS1 shows slightly more rutting than those of AS0. This result is opposite to what was expected. A statistical t-test has been performed to analyse the data (see Table 4.11). The “null hypothesis” (H_0) of this test is: mean rut (AS0) = mean rut (AS1). As the p-value is far from 0.05 (0.56–0.67 according to the number of cycles) whatever the number of cycles, the H_0 hypothesis cannot be rejected. Then, from a statistical point of view, the results of this test do not show any significant difference between tests without anti-sticking (AS0) and tests with anti-sticking (AS1).

A comparison of the standard deviations has also been performed and does not show any significant trend to limit the scattering of results with the anti-sticking procedure. It can, however, be noticed that this procedure tends to decrease the scattering between the left and right specimens during each AS1 test.

It appears that the anti-sticking procedure has no effect statistically, except for slightly decreasing the scattering between left and right specimens. This is interesting because in the rare cases where sticking is observed, it may stop sticking and allow the test to be performed; and in the cases where sticking is not observed, mean results do not seem to be really affected. Few laboratories have observed sticking, only Lab1 and Lab3. For them, the procedure enables the test to be performed to the end.

Considering that specimens were statistically the same for all the laboratories, it was thought that the origin of the sticking could be related to differences between tyres. This could also have an influence on scattering of results.

4.3.2.9 Influence of the Anti-overheating Procedure

This procedure was supposed to be applied to all the tests, at 50 and 60°C. At 60°C, some laboratories have performed these tests while precisely measuring the temperatures inside the specimen, as a function of time through tests. Only these results will be discussed, they reflect the general trend.

Some results of rut depth and temperature as a function of cycles for Lab1, Lab4 and Lab7 are presented in Figs. 4.65, 4.66 and 4.67, respectively.

Firstly, these results first show that none of these three laboratories succeeded in fully following the anti-overheating procedure through the tests. Indeed, according to the “temperature” versus “Nb of cycles” curves, each sudden decrease of temperature is related to a stop of the test and opening of the door of the thermal chamber. This stop enables the temperature to decrease before test is restarted. We can observe that all the participants let temperature exceed the critical limit of 62°C at the end of the tests (after approximately 20 stop procedures). To follow totally the procedure was too much of a constraint for the technicians in charge of the tests and would have been very time consuming. As an example, AS0 lab7 was performed

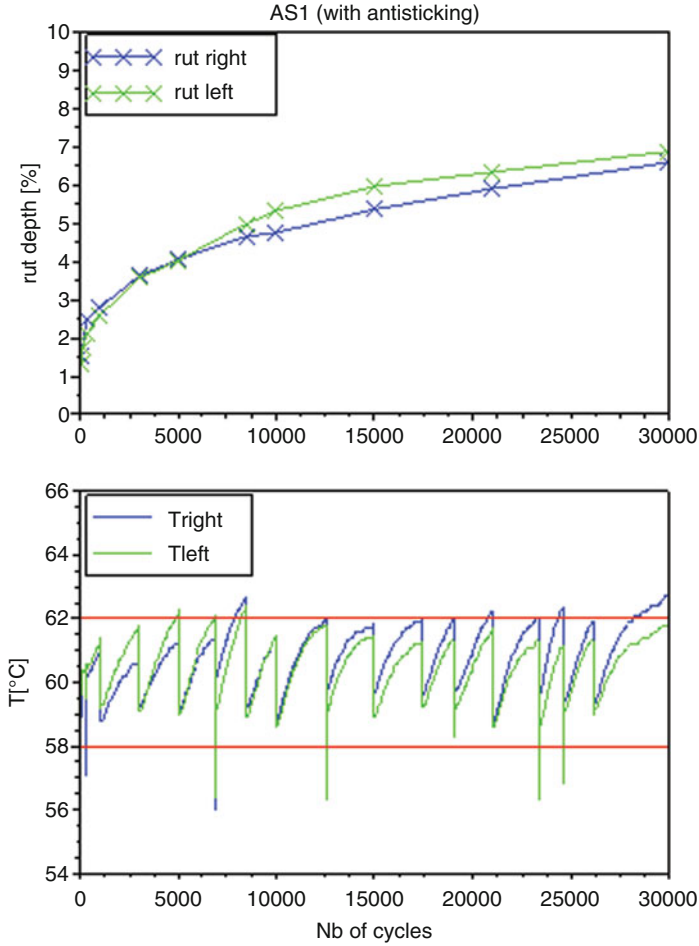


Fig. 4.65 Results of lab01 for a AS1 test. Rut depths and temperatures measured inside the samples as a function of the number of cycles

over 3 weeks. We can also see that even when the procedure was followed, the limit temperature was sometimes slightly exceeded. Technicians could not stay in front of the apparatus, which can explain late reactions. A possible improvement would be to automate the procedure. However, it can be concluded that the procedure is well respected from the beginning and during the main part of the test.

4.3.2.10 Correlation Between Rut and Temperature Increase

Results, presented in Figs. 4.65, 4.66 and 4.67, tend to show a correlation between rutting and temperature increase. Indeed, if left and right rut depths are compared to

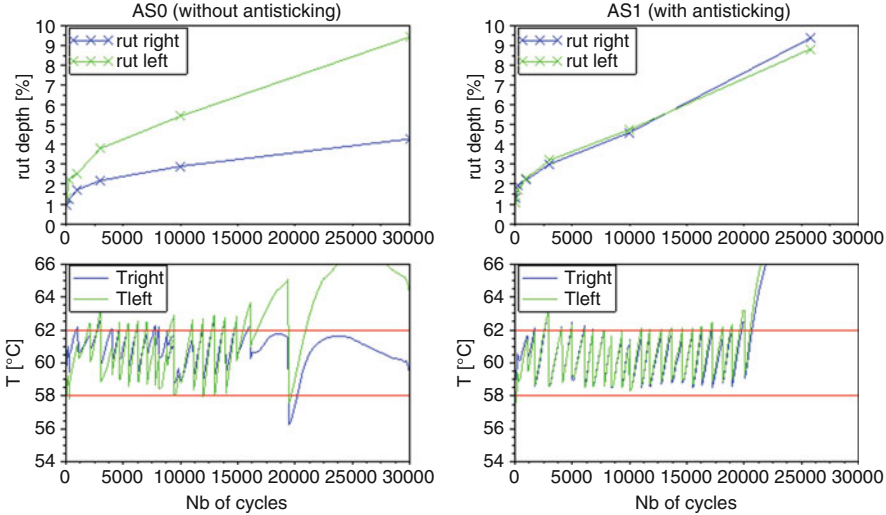


Fig. 4.66 Results of lab4. For both an AS0 and an AS1 test, rut depths and temperatures measured inside the samples as a function of the number of cycles

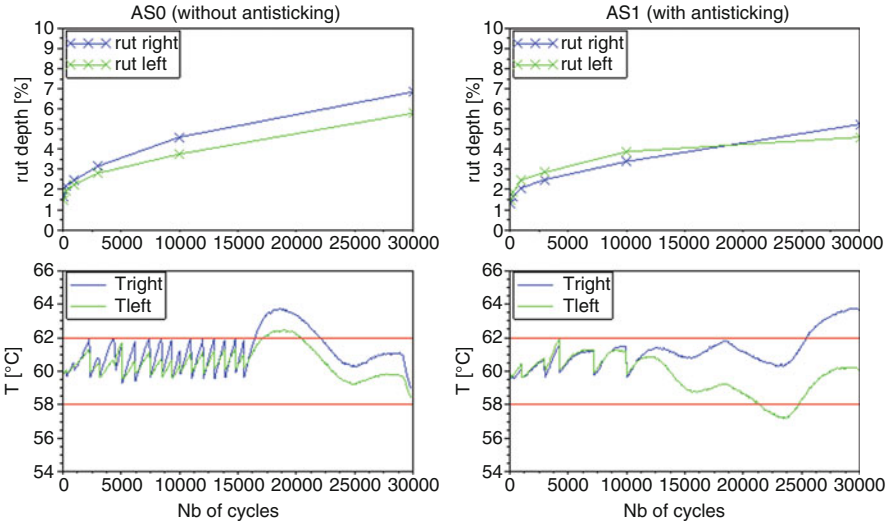


Fig. 4.67 Results of lab7. For both an AS0 and an AS1 test, rut depths and temperatures measured inside the samples as a function of the number of cycles

left and right temperature evolutions, it can be seen for the presented tests, that equal rut depth curves can be related to identical temperature curves (Figs. 4.65 and 4.66 (AS1)). For the others, higher rut depths can be correlated to higher temperatures during tests. For example, during the AS0lab4 test (Fig. 4.66 (AS0),

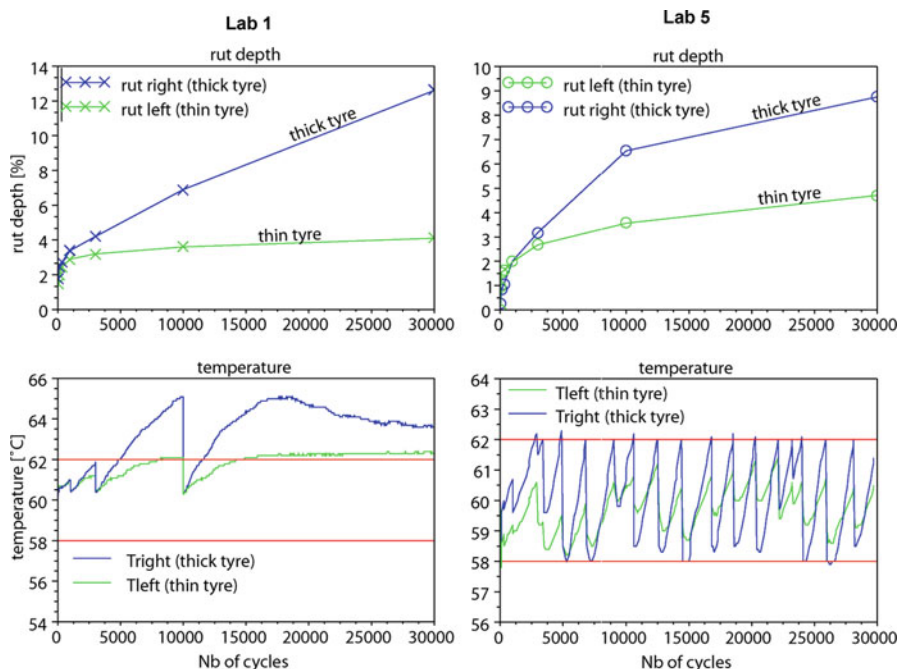


Fig. 4.68 FWT performed on both Lab1 and Lab5 apparatus, equipped with a thick and a thin tyre. Rut depths and temperatures versus number of cycles

peaks of temperature are higher and rutting is more significant, as well as for the test AS0lab7 (Fig. 4.67 (AS0)). At the end of the AS1lab7 test (Fig. 4.67 (AS1)), between 10,000 and 30,000 cycles, the higher temperature can be correlated to an increase of rut depth. Results presented in Fig. 4.68 (in the framework of a study of the influence of tyres) seem to confirm this correlation between the level of temperature and the level of rut depth.

This trend between the increase of temperature and increase of rut depth is well known for bituminous mixtures. It can be explained by the thermo-susceptibility of this material; an increase of temperature during the test leads to a decrease of viscosity and stiffness of material and this then involves more movements of the granular component and more rutting (Dongmo [70]).

4.3.2.11 Are the Tyres Responsible for Scattering?

A difference between tyres has been suggested as a possible cause of the variability of the results. Closed to the end of the campaign, it was found after recheck of experimental conditions that surprisingly, different types of tyres were used among the teams. All the tyres have the same profile and apparently the same dimensions.

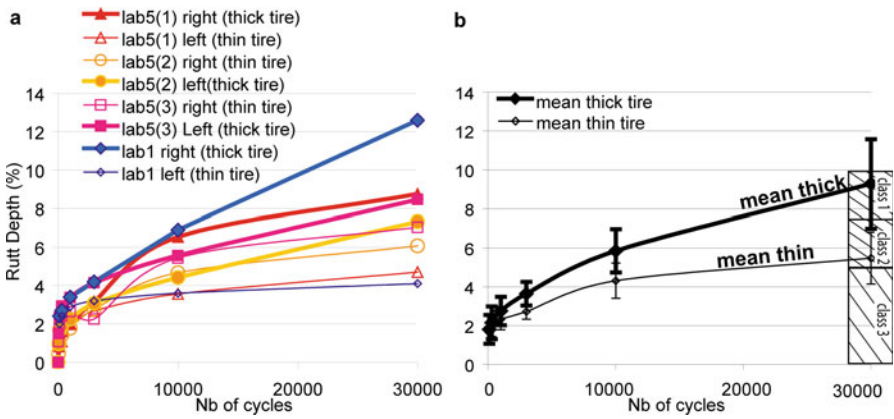


Fig. 4.69 (a) FWTT performed by Lab1 (1 test) and Lab5 (3 tests), equipped with a thick and a thin tyre. (b) Comparison by means values of thin and thick tyres. Performance class considering French design method is also indicated

Table 4.12 Tyres mass comparison between Lab 1 and Lab 2

Lab	1	2
Mass(g)	3,290	1,890

But a difference was seen in the weight and thickness as explained in the following section.

To check this, Lab1 and Lab5 have performed FWT tests, using a “thin tyre” on one side and a “thick tyre” on the other side of the apparatus. Results are presented in Figs. 4.68 and 4.69. In Fig. 4.68, rut depths and temperatures are plotted as a function of the number of cycles. These results are presented separately because they are useful both for this study and also the study on the influence of temperature (see below).

Both rut depth and temperature reach lower levels on the side equipped with the “thin tyre”. Lab5 has carried out three tests in this configuration. Results are summarized in Fig. 4.69a, where it can be seen that rutting is always more significant when the thick tyre is used. Figure 4.69b also shows less scattering (in absolute terms) for tests performed with the thin tyre. These results clearly highlight a strong influence of tyre.

The thick and thin tyres have been analysed in terms of masses, sizes, structures and foot prints. Results are presented in Table 4.12, Figs. 4.70, 4.71 and 4.72.

This comparison highlights an important difference between the two tyres, leading to different diameters and different footprints for a same inflation pressure (0.6 MPa). The tyre from Lab1 has a thicker structure constituted of six layers and the tyre from lab2 has two Layers. Consequently, the tyre from Lab1 is stiffer and shows a smaller footprint. Prints are 70 and 88 cm² for the thick and the thin tyre respectively. In addition, even if such a print cannot give the local intensity of the pressure at a given location on the surface, the less coloured parts of the print can be



Fig. 4.70 Tyres size comparison between Lab 1 and Lab 2

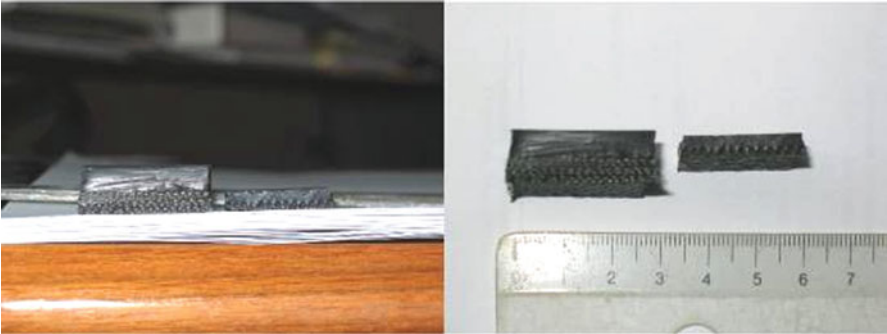


Fig. 4.71 Pictures of tyre sections: Lab 1 (*thick tyre* on the *left side* of both pictures) and Lab 2 (*thin tyre* on the *right side* of both pictures)

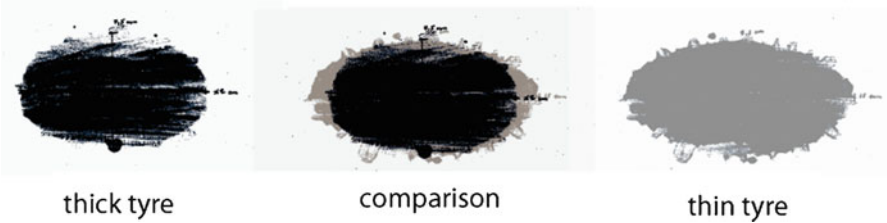


Fig. 4.72 Tyre footprints and superposition (*thin* and *thick tyres*)

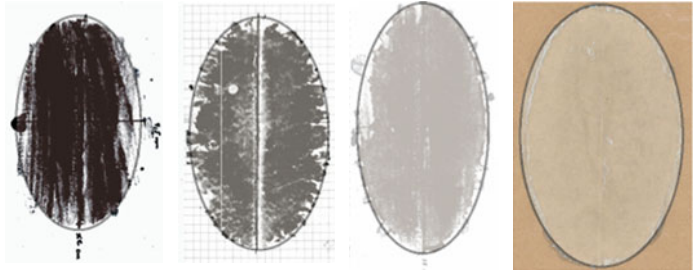
related to low pressure zones, which amplify the level of pressure at the middle of the thick tyre (Fig. 4.72). Thus, the specimen under a thicker tyre is subjected to a more concentrated pressure, which leads to more rutting. This result is quite typical for bituminous mixtures.

Table 4.13 Mass of tyres according to the laboratory

Lab	1	2	3	4	5	6	7
mass(g)	3,290	1,890 or 3,200 ^a	2,750	3,112	3,071	2,150	3,200

^a The two models can be used by Lab02 for rutting tests

Table 4.14 Prints ranked from the smallest to the biggest (same scale)

Laboratories	Lab1	Lab5	Lab2	Lab3
Print				
Print surface [cm ²]	70	85	88	112
Mass [g]	3,290	3,071	1,890	2,750
Mean rut depth at 60°C and 30,000 cycles [%]	6.73	7.06	5.18	6.7

Each laboratory has been asked to assess its tyres. It turned out that there were not only thin and thick tyres, but also intermediate. A comparison between the measured masses, the rut depths and the print surfaces has been performed (see Tables 4.13 and 4.14). This has not allowed a clear demonstration of a direct correlation between these three results: mass ranking, rut depth ranking and print ranking. It can be concluded that different tyres may lead to different results, but the different results are not only due to differences between tyres.

The European standard EN 12697–22 [77] stipulates (in paragraph 6.1.1.1) the pressure inside the tyre, the dimensions (440 × 8) and the track width (80 ± 5 mm). In a note of this standard, “the Trelleborg T522 BV Extra or Special 6 ply type” is considered as “suitable for this test”, but as this is only a note, no laboratory is, however, constrained to use this type of tyre. All the laboratories did use a Trelleborg T522 tyre, but it seems that under this unique reference, two types of tyres (two and six ply) are produced. Since the present study demonstrated that this affects the results, users must be aware of that. It is necessary to make the EN 12697–22 [77] more specific on this point.

4.3.2.12 Conclusion on Wheel Tracking Test Campaign “B”

Under the auspices of the RILEM TC 206-ATB TG3, this experimental programme has been carried out in order to evaluate the capability of the French Wheel Tracking Test to characterise the rutting behaviour of asphalt concrete containing bitumen highly modified with polymer. A particular plastomer of EVA type has been chosen to modify the bitumen. The Round Robin Test performed has highlighted, for such test on PmB-AC, a reproducibility of results comparable to those obtained in other Round Robin Tests on classical materials (EN 12697–22 [77]; Poirier [110]).

Two specific procedures have been assessed: an anti-overheating and an anti-sticking procedure. From a practical viewpoint, it was observed in previous studies that the anti-sticking procedure could be helpful to avoid sticking problems between the tyre and the sample. Our analysis has shown that such a procedure has statistically no influence on the results of the performed campaign. Only a slight positive effect which consists in a trend to decrease the scattering between the left and right side results on a given apparatus. Our conclusion is that it should be a free choice to use this anti-sticking procedure. It could be necessary if a strong sticking phenomenon appears.

An anti-overheating procedure has also been developed to be followed by all the laboratories during this RRT. Such a procedure prevents the tests from being partially run outside of the standard in terms of temperature control. Unfortunately, in some cases, like tests on our AC-PmB, respecting this procedure strictly is quite difficult, as it can increase the test duration to more than 2 weeks and necessitates many actions from a technician. Therefore, this apparatus could be improved by automating the stop and start function to turn off when the standard temperature is exceeded and then restart when the minimal requested temperature is reached.

It has also been revealed that a large scattering was caused by the difference between the tyres used on the different apparatus. Despite a common main reference number, tyres can have different structures and thicknesses. Then, it is important to control structure of each tyre, as well as its print in terms of length and width (and not only the width as mentioned in the standard). In addition, problems of sticking could in some cases probably be solved by using tyre as advised by the standard.

The sample preparation can also be the cause of the scattering. During this programme some of the manufactured concrete slabs were over-compacted, which creates “outliers” that were not considered for the analysis. Finally, studying the rutting problem on this type of apparatus needs great care, due to the strong heterogeneity, the complexity of the loading, and the poorly controlled boundary conditions.

Then, if the previous aspects (strict temperature control, tyre characteristics, sticking prevention and specimen preparation) are respected, reproducibility could be strongly improved. This remark is true for any kind of asphalt concretes made with pure binders or modified binders.

4.4 Modeling and Finite Elements Calculation

In parallel with the round robin tests campaign, a sub-group was focused on modeling and simulation using Finite Elements Method. The scope of the sub-group was to build numerical framework based on finite element (FE) simulations for rutting prediction of asphaltic layers.

Before performing permanent (viscoplastic) deformations simulation, the work has first concentrated on simulating the reversible viscoelastic stresses and strains in wheel tracking machines without permanent deformation.

In order to make it possible to compare results between each team, some assumptions and modeling conditions were imposed. Among them, the linear viscoelastic (LVE) material behaviour was modeled by a generalised Kelvin-Voigt model, with fixed and given parameters. It appeared rapidly that the corresponding (or equivalent) generalised Maxwell model was used instead of the Kelvin-Voigt model in some of the considered FE code. The set of parameters for the generalized Maxwell model was required to provide the same results.

Thus, both generalised Kelvin-Voigt and Maxwell models are first presented as well as their calibration. A method is proposed to determine generalised Maxwell parameters from generalised Kelvin-Voigt parameters so that both models are equivalent.

Secondly, the first FE simulations are described and their results are presented.

4.4.1 Linear Viscoelastic Models and Their Calibration

Bituminous materials have a linear viscoelastic behaviour for small strain level (Doubbaneh [72]; Airey et al. [2]; Olard and Di Benedetto [103]; Di Benedetto and Corté [62]; Delaporte et al. [51]; Di Benedetto et al. [67]; Chailleux et al. [36]; Pouget et al. [111, 112]). If these materials are considered as isotropic, the linear viscoelastic properties are characterised by the complex modulus E^* and the complex Poisson's ratio ν^* for the three-dimensional description. It has been shown (Di Benedetto et al. [63]) that the Poisson's ratio is a complex number, which is also frequency and temperature dependent. This dependency should be taken into account to get better simulations. Nevertheless, in order to simplify the Finite Element calculation, it was chosen to consider the Poisson's ratio as a constant real number. In the following, it has been classically taken equal to 0.35.

Different models have been developed and proposed to obtain the complex modulus of bituminous materials. Different types of models are proposed, some are empirical such as the Witczak model (Witczak and Fonseca [130]) and the Hirsch model (Christensen et al. [39]), which predict the mix modulus from binder and mix properties. Others are based on analogical elementary models such as the Maxwell model (Fig. 4.73), the Kelvin-Voigt model (Fig. 4.73), the Huet-Sayegh model (Huet [90]) and the 2S2P1D model (two springs, two parabolic elements and

Fig. 4.73 Representation of a Kelvin-Voigt model (*left*) and Maxwell model (*right*):
 E stiffness of the spring,
 η viscosity of the dashpot

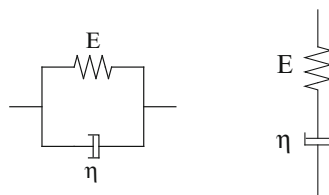
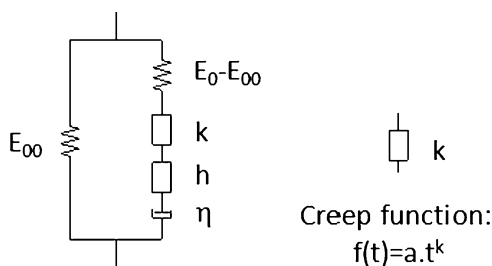


Fig. 4.74 Representation of 2S2P1D model in 1D (*left*) and parabolic element (*right*)



one dashpot, Fig. 4.74), which is a generalisation of the Huet-Sayegh model (Di Benedetto et al. [61, 64]; Olard and Di Benedetto [103]). In this study, the 2S2P1D model is used as a continuous spectrum model to calibrate the Generalised Kelvin-Voigt and Generalised Maxwell models.

In the next section, these models and their calibration are described. The linear viscoelastic characterization and modeling of the mix used for the round robin on the French WTT, called campaign B (see Sect. 4.3.2) is then proposed.

4.4.1.1 Description of Used LVE Models

The 2S2P1D model developed at the DGCB laboratory of ENTPE is well adapted to simulate the linear viscoelastic behaviour of bituminous materials (mixes, mastics and bitumen) as previously shown in different publications (Olard and Di Benedetto [103]; Di Benedetto et al. [61, 64]). Besides, this model is easy to calibrate due to the small number of constants to set up. However the 2S2P1D model has a continuous spectrum and can hardly be used in finite element calculations. This explains why the global strategy developed at ENTPE is to use 2S2P1D model for the calibration of Generalised Kelvin-Voigt model (GKV) with “ n ” elements (Fig. 4.75). The number of elements “ n ” is the choice of the user. It is clear that the quality of fitting improves with the number of elements. Some FEM software use the Generalised Maxwell model (GM) (Fig. 4.76). It is thus an important feature for structure calculation to be able to obtain the set of parameters of GKV or GM, allowing good description of the bituminous materials behaviour on the wide range of temperatures and frequencies existing on site.

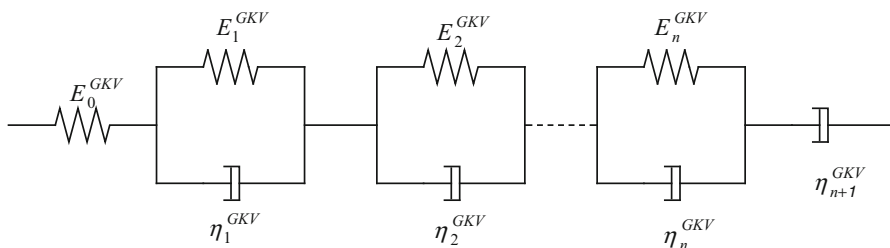


Fig. 4.75 Representation of a Generalized Kelvin-Voigt (GKV) model (for *solid* type materials)

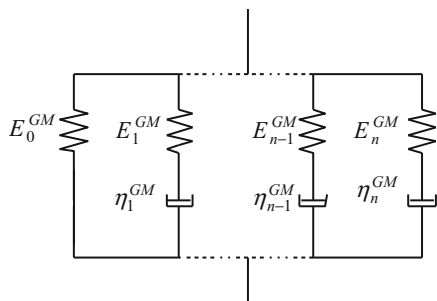


Fig. 4.76 Representation of a Generalised Maxwell (GM) model (for *solid* type materials)

2S2P1D Model in One Dimension

The 2S2P1D model (one dimension formulation), developed at DGCB laboratory, is a generalization of the Huet-Sayegh model (Huet [90]). This model is based on a simple combination of physical elements (spring, dashpot and parabolic creep element) (Fig. 4.74). This model is able to simulate the three dimensional behaviour for any bituminous materials: binder, mastic or mix (Di Benedetto et al. [61, 64]). The model applicable in the three dimensional case is described in Di Benedetto et al. [64]. In this work, the 2S2P1D model is restrained to the one dimensional case.

At a given temperature, the complex modulus of the 2S2P1D model is given by Eq. (4.19)

$$E_{2S2P1D}^*(\omega) = E_{00} + \frac{E_0 - E_{00}}{1 + \delta(j\omega\tau)^{-k} + (j\omega\tau)^{-h} + (j\omega\beta\tau)^{-1}} \quad (4.19)$$

where:

j is the complex number defined by $j^2 = -1$
 ω is the pulsation, $\omega = 2\pi f$, (f is the frequency)
 k, h : constants such as $0 < k < h < 1$
 δ : constant

E_{00} the static modulus when $\omega \rightarrow 0$

E_0 the glassy modulus when $\omega \rightarrow \infty$

η : Newtonian viscosity of the dashpot, $\eta = (E_0 - E_{00})\beta\tau$

τ : characteristic time, whose value varies only with temperature

If the time-temperature superposition principle is verified it becomes:

$$\tau(T) = a_T(T) \cdot \tau_0 \quad (4.20)$$

where a_T is the shift factor at the temperature T . $\tau = \tau_0$ at the reference temperature T_{ref} .

At a given temperature, seven constants (E_{00} , E_0 , δ , k , h , β , τ_0) are required to completely characterise the linear viscoelastic properties of the considered material.

When temperature effect is considered, if the time-temperature superposition principle is verified, τ evolution is approximated by a WLF type law (Williams et al. [127]):

$$\log(a_T) = -\frac{C_1(T - T_{ref})}{C_2 + T - T_{ref}} \quad (4.21)$$

The number of constants amounts to nine including the two WLF constants (C_1 and C_2).

Generalised Kelvin-Voigt and Generalised Maxwell Models

The Generalised Kelvin-Voigt (GKV) (Fig. 4.75) and Generalised Maxwell (GM) (Fig. 4.76) models consist of a finite number “n” of models of Maxwell and Kelvin-Voigt bodies (Fig. 4.73) in parallels or in series. It can be shown (Mandel [96]) that in the one-dimensional case, both models can describe any type of linear viscoelastic behaviour eventually with “n” tending toward infinity.

In addition for a same number of elements “n”, and with a correct calibration of constants, E_i^{GKV} , η_i^{GKV} for GKV model and constants E_i^{GM} , η_i^{GM} for GM model exactly the same rheological behaviour can be obtained.

The complex modulus of a generalised Kelvin-Voigt (GKV) model (Fig. 4.75) is given by:

$$E_{GKV}^*(\omega) = \left(\frac{1}{E_0^{GKV}} + \sum_{i=1}^n \frac{1}{E_i^{GKV} + j\omega\eta_i^{GKV}} \right)^{-1} \quad (4.22)$$

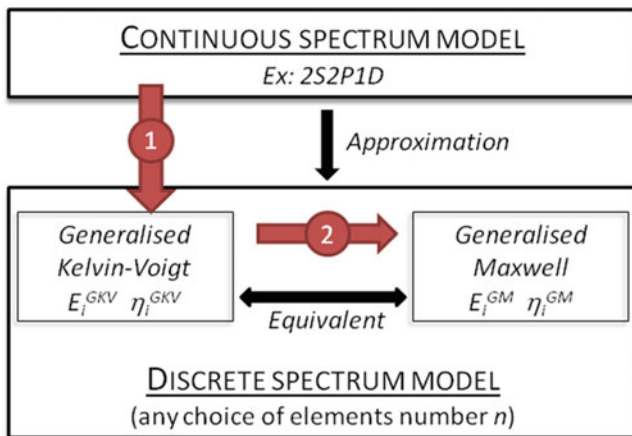


Fig. 4.77 Principle of proposed calibration method: Step 1, approximation of any continuous spectrum model by Generalised Kelvin-Voigt model (GKV) – Step 2, calibration of the equivalent Generalised Maxwell model (GM)

The complex modulus of a Generalised Maxwell (GM) model (Fig. 4.76) is given by:

$$E_{GM}^*(\omega) = E_0^{GM} + \sum_{i=1}^n \frac{j\omega\eta_i^{GM}E_i^{GM}}{E_i^{GM} + j\omega\eta_i^{GM}} \quad (4.23)$$

4.4.1.2 Calibration Method

The purpose of this section is to present the developed method to approximate a linear viscoelastic model by a Generalised Kelvin-Voigt (GKV) model or a Generalised Maxwell (GM) model with “n” bodies, where “n” is the choice of the user (Fig. 4.77).

In a first step, the constants of the generalised Kelvin-Voigt are determined to approach the linear viscoelastic 2S2P1D model with continuous spectrum. The second step consists in calculating the constants of the generalised Maxwell from the constants of the Generalised Kelvin-Voigt model (Tiouajni et al. [122]).

Considering bituminous materials (bitumen, mastics and mixes), the 2S2P1D model has already proven its ability to model their behaviour. In the next section, this model will be used and approximated by both GKV and GM models.

Approximation of a Linear Viscoelastic Model by a GKV Model

The calibration of generalised Kelvin-Voigt model consists of calculating the modulus E_i^{GKV} of the $n + 1$ springs and the viscosity and η_i^{GKV} of the n dashpots giving a behaviour as “close” as possible than the LVE behaviour to be

approximated. In the following, the 2S2P1D model is chosen as the reference LVE model to be approximated (the method could be applied to any other model).

The optimisation process is developed in the frequency domain. $E_{2S2P1D}^*(\omega)$ is the complex modulus of the linear viscoelastic 2S2P1D model to be approximated. E_{00} is the static modulus when ω tends toward 0 and E_0 the glassy modulus when ω tends toward ∞ . The problem is then to obtain $E_{GKV}^*(\omega)$ as “close” as possible as $E_{ref}^*(\omega)$ for any values of pulsation ω .

The optimisation procedure begins with the free choice of the number of bodies “ n ” as well as the frequency domain for the optimisation $[f_{min}, f_{max}]$. For f lower than f_{min} , the complex modulus is constant and equal to E_{00} . Respectively, for f higher than f_{max} , the complex modulus is constant and equal to E_0 .

The following approach is used in order to determine the value of the constants E_i^{GKV} ($n + 1$ values) and η_i^{GKV} (n values) of the GKV model.

First, the rigidity of the first spring E_0^{GKV} is determined. This value corresponds to the rigidity of the linear viscoelastic body when the pulsation ω tends toward infinity.

$$E_0^{GKV} = E_0 \quad (4.24)$$

Then, for general calculation of the model constants (η_i^{GKV} and E_i^{GKV}), a recurrence procedure is used.

“ n ” frequencies f_i are chosen in the interval $[f_{min}, f_{max}]$. These frequencies are equally assigned on a log scale. It becomes:

$$\frac{f_i}{f_{i+1}} = 10^{\frac{\log(f_{max}) - \log(f_{min})}{n}} \quad \text{and} \quad f_n = f_{min} \quad (4.25)$$

These frequencies are used to set the chosen characteristic time (ξ_i^{GKV}) of each of the simple Kelvin-Voigt bodies by using Eq. (4.26):

$$\xi_i^{GKV} = \frac{\eta_i^{GKV}}{E_i^{GKV}} = \frac{1}{2\pi f_i} \quad (4.26)$$

The recurrence procedure has “ n ” steps as the number of simple Kevin Voigt bodies “ n ”.

At the step “ k ”, the constants E_i^{GKV} and η_i^{GKV} for $1 \leq i \leq k - 1$ are supposed known and the calculation of E_k^{GKV} and η_k^{GKV} is made.

Considering a cyclic loading with a pulse equal to $\omega_k (= 1/\xi_k^{GKV})$, the GKV model can be approximated by only the first k bodies (Fig. 4.78). The bodies with $i > k$ having ξ_i smaller than ξ_k can be considered as “infinitely rigid”.

For $\omega = \omega_k$, the complex modulus of the approximated GKV model is given by:

$$E_{GKV}^*(\omega_k) = \left(\frac{1}{E_0^{GKV}} + \sum_{i=1}^k \frac{1}{E_i^{GKV} + j\omega_k \eta_i^{GKV}} \right)^{-1} \quad (4.27)$$

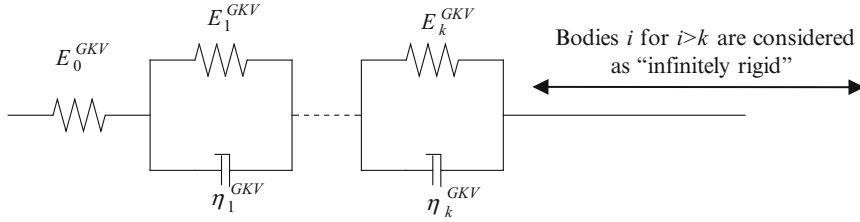


Fig. 4.78 Approximation of Generalized Kelvin-Voigt model for a solicitation pulsation equal to $\omega_k (=1/\xi_k^{GKV})$

Then the problem consists of finding the value E_k^{GKV} of the body “k” that gives at the pulsation ω_k , a modulus as close as possible to the modulus of the linear viscoelastic body to be approximated (Eq. 4.26).

$$E_{GKV}^*(\omega_k) \approx E_{2SP1D}^*(\omega_k) \quad (4.28)$$

Combining Eqs. (4.27) and (4.28), the following is obtained:

$$\frac{1}{E_k^{GKV}} (1 - j\omega_k \xi_k^{GKV}) \approx \left(\frac{1}{E_k'} - j \frac{1}{E_k''} \right) \quad (4.29)$$

With (Re is the real part and Im the imaginary part of the complex number):

$$E_k' = \left((1 + (\omega_k \xi_k^{GKV})^2) \times \left(\frac{\text{Re}(E_{ref}^*(\omega_k))}{|E_{ref}^*(\omega_k)|^2} - \frac{1}{E_0} - \sum_{i=1}^{k-1} \frac{1}{E_i^{GKV} (1 + j\omega_i \xi_i^{GKV})} \right) \right)^{-1} \quad (4.30)$$

$$E_k'' = \left((1 + (\omega_k \xi_k^{GKV})^2) \times \left(\frac{\text{Im}(E_{ref}^*(\omega_k))}{|E_{ref}^*(\omega_k)|^2} - \sum_{i=1}^{k-1} \frac{\omega_k \xi_i^{GKV}}{E_i^{GKV} (1 + j\omega_k \xi_i^{GKV})} \right) \right)^{-1} \quad (4.31)$$

To set the value of E_k^{GKV} , the absolute value of both terms of Eq. (4.29) are chosen as equal, for the pulsation $\omega_k = 1/\xi_k^{GKV}$.

It finally comes:

$$E_k^{GKV} = \frac{1}{\sqrt{2}} \times \sqrt{E_k'^2 + E_k''^2} \quad (4.32)$$

The viscosity η_k^{GKV} is obtained using Eq. (4.33).

$$\eta_k^{GKV} = \zeta_k^{GKV} E_k^{GKV} \quad (4.33)$$

This method for the optimisation of the GKV model is implemented in an excel sheet that is fast and easy to use.

Calibration of Generalised Maxwell (GM) from GKV

In order to calibrate the Generalised Maxwell model, it is first necessary to obtain the calibrated parameters of the Generalised Kelvin-Voigt model. The complex moduli of both models (GM and GKV) are given by the following expressions (Eqs. 4.34 and 4.35). The case of solid like materials is considered.

$$E_{GM}^*(\omega) = E_0^{GM} + \sum_{i=1}^n \frac{j\omega\eta_i^{GM} E_i^{GM}}{E_i^{GM} + j\omega\eta_i^{GM}} \quad (4.34)$$

$$E_{GKV}^*(\omega) = \left(\frac{1}{E_0^{GKV}} + \sum_{i=1}^n \frac{1}{E_i^{GKV} + j\omega\eta_i^{GKV}} \right)^{-1} \quad (4.35)$$

The equality between the complex modulus of the GKV and GM bodies when the frequency tends towards zero gives the first parameter:

$$E_0^{GM} = \left(\sum_{i=0}^n \frac{1}{E_i^{GKV}} \right)^{-1} \quad (4.36)$$

Equations (4.34) and (4.35) can be written in a single fraction, as follows:

$$E_{GM}^*(\omega) = \frac{E_0^{GM} \prod_{i=1}^n (E_i^{GM} + j\omega\eta_i^{GM}) + \sum_{i=1}^n j\omega\eta_i^{GM} E_i^{GM} \prod_{\substack{k=1 \\ k \neq i}}^n (E_k^{GM} + j\omega\eta_k^{GM})}{\prod_{i=1}^n (E_i^{GM} + j\omega\eta_i^{GM})} \quad (4.37)$$

$$E_{GKV}^*(\omega) = \frac{\prod_{i=1}^n (E_i^{GKV} + j\omega\eta_i^{GKV})}{\frac{1}{E_0^{GKV}} \prod_{i=1}^n (E_i^{GKV} + j\omega\eta_i^{GKV}) + \sum_{i=1}^n \prod_{\substack{k=1 \\ k \neq i}}^n (E_k^{GKV} + j\omega\eta_k^{GKV})} \quad (4.38)$$

The calibration of the GM model consists of finding the constants in order to obtain the same complex modulus as the GKV model. Then Eqs. (4.37) and (4.38) should give the same result for all pulsations. This can be achieved only if the polynomial expressions of the denominators of Eqs. (4.37) and (4.38) have the same roots. The roots of the denominator in Eq. (4.37) for the GM model are easily determined:

$$j \frac{E_i^{GM}}{\eta_i^{GM}} \quad (4.39)$$

The constants of GKV model are supposed known, thus the roots of the denominator in Eq. (4.37) for the GKV model can be determined by a numerical resolution. These roots are noted R_i with $1 \leq i \leq n$. The identification between the roots gives:

$$\frac{E_i^{GM}}{\eta_i^{GM}} = \text{Im}(R_i) \quad (4.40)$$

By inserting Eq. (4.40) in Eq. (4.34), the complex modulus of Generalised Maxwell model is given by:

$$E_{GM}^*(\omega) = E_0^{GM} + \sum_{i=1}^n \frac{j\omega}{\text{Im}(R_i) + j\omega} E_i^{GM} \quad (4.41)$$

To finish the calibration of GM model, the equality between the complex modulus of GM model (Eq. (4.41) and the complex modulus of GKV model Eq. (4.35)) is written for “ n ” appropriate pulsations. The values of pulsations are:

$$\omega_i = \frac{E_i^{GM}}{\eta_i^{GM}} = \text{Im}(R_i) \quad (4.42)$$

The obtained linear system of equation has to be solved:

$$\begin{pmatrix} \frac{j\omega_1}{\text{Im}(R_1) + j\omega_1} & \cdot & \cdot & \frac{j\omega_1}{\text{Im}(R_n) + j\omega_1} \\ \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot \\ \frac{j\omega_n}{\text{Im}(R_1) + j\omega_n} & \cdot & \cdot & \frac{j\omega_n}{\text{Im}(R_n) + j\omega_n} \end{pmatrix} \begin{pmatrix} E_1^{GM} \\ \cdot \\ \cdot \\ E_n^{GM} \end{pmatrix} = \begin{pmatrix} E_{GKV}^*(\omega_1) - E_0^{GM} \\ \cdot \\ \cdot \\ E_{GKV}^*(\omega_n) - E_0^{GM} \end{pmatrix} \quad (4.43)$$

The resolution of the linear system gives the value of the “ n ” rigidity of springs E_i^{GM} . The value of the viscosities is then given by Eq. (4.44) (from Eq. (4.40)).

$$\eta_i^{GM} = \frac{E_i^{GM}}{\text{Im}(R_i)} \quad (4.44)$$

Table 4.15 Parameters of one dimensional 2S2P1D model calibrated for bituminous mix of campaign B on FWTT

E_{00} (MPa)	E_0 (MPa)	k	H	δ	τ_E (s)	β
180	42,000	0.23	0.50	2.00	0.04	400

Then for a solid like material ($\eta_{n+1}^{GM} = \infty$) the constant values of the GM model are obtained. This method allows not only to calibrate the Generalised Maxwell model but to obtain two models (GM and GKV) which give a unique viscoelastic behaviour. This will be applied to the bituminous mix of campaign B (see Sect. 4.3.2).

4.4.1.3 Application to the Bituminous Mix of Campaign B (Sect. 4.3.2)

The linear viscoelastic behaviour of the mix, which has been used for campaign B (Sect. 4.3.2) on the French Wheel Tracking round-robin test, was characterised using complex modulus test (Sect. 4.3.2.5). The presented modeling and calibration method is used on this mix (see Sect. 4.3.2.2 for description of the mix).

Modeling with One Dimensional 2S2P1D Model

The results of the complex modulus tests are presented in Sect. 4.3.2.5. The 2S2P1D model has been calibrated and the simulations are also plotted with experimental data. It should be noted that only the complex modulus is modeled as the one dimensional 2S2P1D model is used. The three dimensional 2S2P1D is able to simulate the complex Poisson's ratio as well, but it is not presented here.

The calibrated parameters of the one dimensional 2S2P1D model are given in Table 4.15.

The complex modulus results are presented in Cole-Cole and Black's space in Fig. 4.79. The data show clearly the influence of the polymer. Nevertheless, in the first approximation, it seems possible to define a unique curve, which shows that the time-temperature superposition principle is also applicable. Then, master curves are built for complex modulus at reference temperature $T_{ref} = 11.3^\circ\text{C}$ and compared with 2S2P1D model (see Fig. 4.80).

Calibration of Generalised Kelvin-Voigt and Maxwell Model

From the fitted 2S2P1D model, it is possible to calibrate the Kelvin-Voigt and Maxwell model with the proposed procedure. As explained earlier, two preliminary choices are required:

- the number of elements is $n = 19$
- the frequency range for calibration is $f \in [1E-7, 1E-12]$

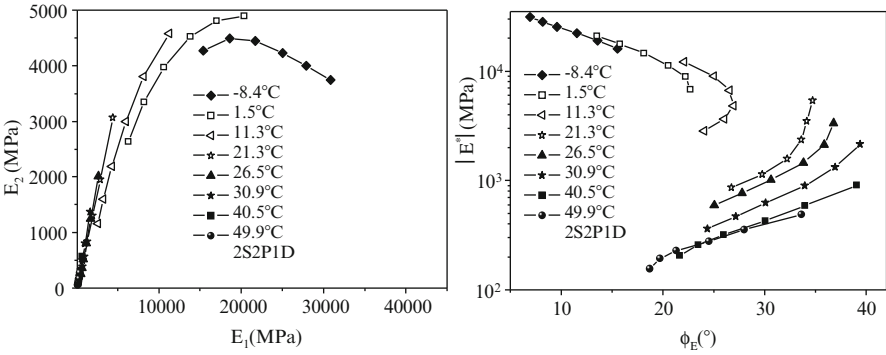


Fig. 4.79 Complex modulus of tested bituminous mix (campaign B on FWTT) in Cole-Cole plane and Black space: experimental results and 2S2P1D modeling

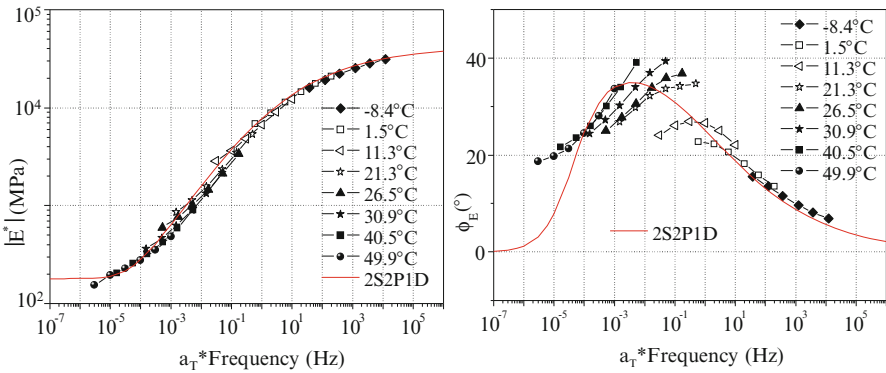


Fig. 4.80 Complex modulus master curves of tested bituminous mix (campaign B on FWTT) at reference temperature $T_{\text{ref}} = 11.3^\circ\text{C}$: experimental results and 2S2P1D modeling

The obtained calibration parameters for generalised Kelvin-Voigt and generalised Maxwell models are given in Table 4.16.

Some comparison between 2S2P1D, Kelvin-Voigt and Maxwell models are presented in Fig. 4.81. As no difference appears between these three models for the complex modulus master curve, results are only presented in the Cole-Cole plane and Black space. In these axes, Maxwell and Kelvin-Voigt may be distinguished from the 2S2P1D model. If the chosen number of elements n increased, the differences would become even smaller. In order to differentiate complex modulus calculated with the Kelvin-Voigt model from the one calculated with Maxwell model, the absolute value of the difference between their real part and their imaginary part are plotted as a function of frequency in Fig. 4.82. The obtained values are negligible compared with the complex modulus value. This shows that the one dimensional Kelvin-Voigt and Maxwell models are equivalent. The

Table 4.16 Parameters of one dimensional Kelvin-Voigt and Maxwell models calibrated for bituminous mix of campaign B on FWTT

Kelvin-Voigt model				Maxwell model			
<i>E0</i>	4.20E + 04			<i>E0</i>	1.76E + 02		
<i>E1</i>	5.55E + 06	$\eta1$	1.06E-05	<i>E1</i>	3.17E + 02	$\eta1$	6.01E-10
<i>E2</i>	4.55E + 06	$\eta2$	8.70E-05	<i>E2</i>	3.78E + 02	$\eta2$	7.16E-09
<i>E3</i>	3.54E + 06	$\eta3$	6.76E-04	<i>E3</i>	4.77E + 02	$\eta3$	9.00E-08
<i>E4</i>	2.08E + 06	$\eta4$	3.97E-03	<i>E4</i>	7.91E + 02	$\eta4$	1.48E-06
<i>E5</i>	1.22E + 06	$\eta5$	2.33E-02	<i>E5</i>	1.29E + 03	$\eta5$	2.38E-05
<i>E6</i>	7.08E + 05	$\eta6$	1.35E-01	<i>E6</i>	2.04E + 03	$\eta6$	3.69E-04
<i>E7</i>	4.08E + 05	$\eta7$	7.79E-01	<i>E7</i>	3.11E + 03	$\eta7$	5.43E-03
<i>E8</i>	2.30E + 05	$\eta8$	4.39E + 00	<i>E8</i>	4.45E + 03	$\eta8$	7.39E-02
<i>E9</i>	1.26E + 05	$\eta9$	2.40E + 01	<i>E9</i>	5.79E + 03	$\eta9$	8.90E-01
<i>E10</i>	6.54E + 04	$\eta10$	1.25E + 02	<i>E10</i>	6.65E + 03	$\eta10$	9.21E + 00
<i>E11</i>	3.17E + 04	$\eta11$	6.06E + 02	<i>E11</i>	6.39E + 03	$\eta11$	7.71E + 01
<i>E12</i>	1.41E + 04	$\eta12$	2.69E + 03	<i>E12</i>	4.92E + 03	$\eta12$	5.06E + 02
<i>E13</i>	5.75E + 03	$\eta13$	1.10E + 04	<i>E13</i>	2.97E + 03	$\eta13$	2.64E + 03
<i>E14</i>	2.19E + 03	$\eta14$	4.19E + 04	<i>E14</i>	1.43E + 03	$\eta14$	1.14E + 04
<i>E15</i>	8.21E + 02	$\eta15$	1.57E + 05	<i>E15</i>	5.81E + 02	$\eta15$	4.50E + 04
<i>E16</i>	3.91E + 02	$\eta16$	7.47E + 05	<i>E16</i>	2.08E + 02	$\eta16$	1.88E + 05
<i>E17</i>	9.50E + 02	$\eta17$	1.81E + 07	<i>E17</i>	1.98E-01	$\eta17$	3.77E + 05
<i>E18</i>	1.84E + 04	$\eta18$	3.51E + 09	<i>E18</i>	7.25E-01	$\eta18$	1.37E + 05
<i>E19</i>	5.71E + 04	$\eta19$	1.09E + 11	<i>E19</i>	3.60E + 01	$\eta19$	5.66E + 05

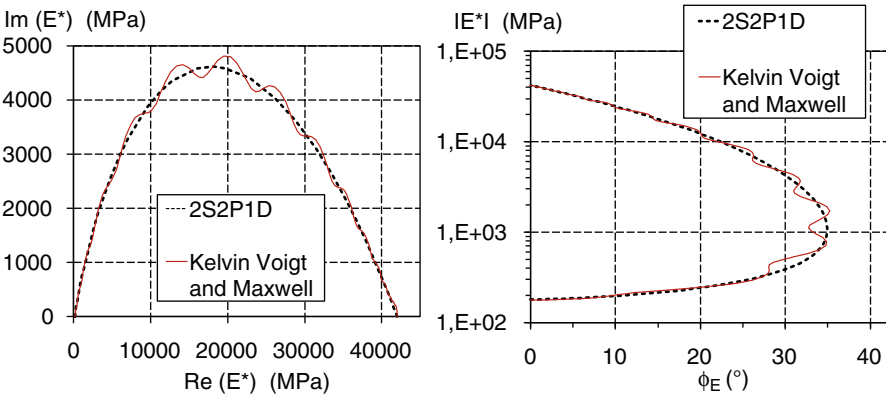


Fig. 4.81 Comparison of 2S2P1D, Kelvin-Voigt and Maxwell modeling in Cole-Cole plane and Black space for complex modulus of tested bituminous mix (campaign B with FWTT) (no differences may be seen between the Kelvin-Voigt and Maxwell model)

proposed procedure gives a reliable method to obtain the constants of the Maxwell model from the constants of the Kelvin-Voigt model, whatever the number of elements n .

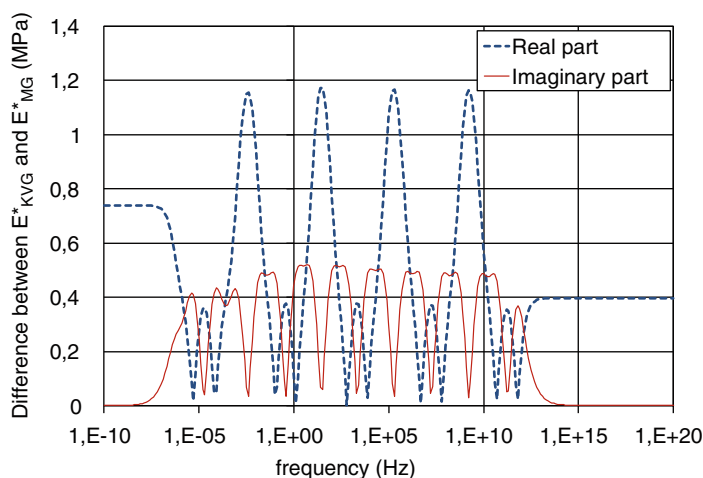


Fig. 4.82 Absolute value of real part and imaginary part of the difference between complex modulus calculated with Kelvin-Voigt model (E_{KVG}^*) and Maxwell model (E_{MG}^*), calibrated for tested bituminous mix (campaign B on FWTT)

4.4.2 *Linear Viscoelastic Finite Elements Calculation of French Wheel Tracking Test Considering French Classical Mix “BBSG”*

Each participant uses their own FE software and their own modeling framework. However, to enable the comparison between the results from all the teams some assumptions and choices have been made to fix the general simulation conditions.

The five participants are listed below:

- ENTPE/Lyon University (France)
- LCPC (France)
- Limoges University (France)
- IBDiM/Warsaw University (Poland)
- Sintef (Norway)

The common assumption required performing the simulations and the obtained results are presented.

4.4.2.1 Description of the Finite Elements Analysis

Different finite elements codes were used for the analysis. Some common elements, necessary to perform the analysis, are given in this section. Simple calculations were first carried out by the involved teams in order to compare obtained results. These preliminary verifications are not described here. The focus is on two simulations involving the geometry of the French wheel tracking test.

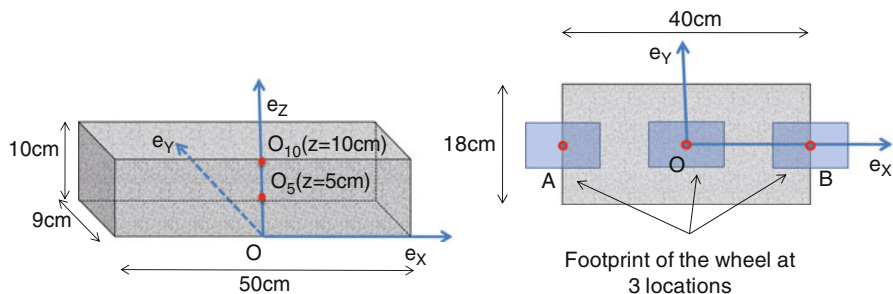


Fig. 4.83 Geometry of the modeled half-slab used in French wheel tracking test (with position of points O_5 and O_{10} for analysis) and path of the wheel on a real slab, extreme position (A and B)

Geometry and Mesh

The slab of a French wheel tracking test is modeled in 3D. The real slabs are 40 cm long, 18 cm wide and 10 cm deep. During the test, the wheel rolls from one side to the other. At both extremities (points A in B in Fig. 4.83), half of the wheel loads onto the mould and the other half on the slab. In order to take into account this effect, the length of the modeled slab is increased. As the problem is symmetric along the wheel path, only half of the slab need be meshed. Then, the modeled slab is 50 cm long, 9 cm wide and 10 cm deep (see Fig. 4.83).

Boundary Conditions

Two sets of boundary conditions are considered:

- Set A: the slab is totally clamped in the mould. Rigid boundaries are applied at ($x = \pm 25$ cm), ($y = +9$ cm) and the bottom ($z = 0$ cm).
- Set B: the contact on the lateral surface between the slab and the mould is a perfect slide. Only the normal displacement is blocked ($U \cdot n = 0$) on the lateral surface ($U \cdot e_x = 0$ at $x = \pm 25$ cm, $U \cdot e_y = 0$ at $y = +9$ cm). The bottom of the slab is still totally clamped ($U = 0$ at $z = 0$).

Loading

The tyre print on the slab is modeled by a rectangle 10 cm (x direction) and 8 cm (y direction). As only half slab is considered, the modeled print is only 4 cm wide. The contact pressure is 1 MPa.

Two cases are modeled. In the first one, the wheel is static in the centre of the slab. The applied load is a function of time as presented in Fig. 4.84.

In the second the wheel moves as in the French wheel tracking tests. The wheel starts on one side of the slab (point A in Fig. 4.83) and moves in a sinusoidal way at

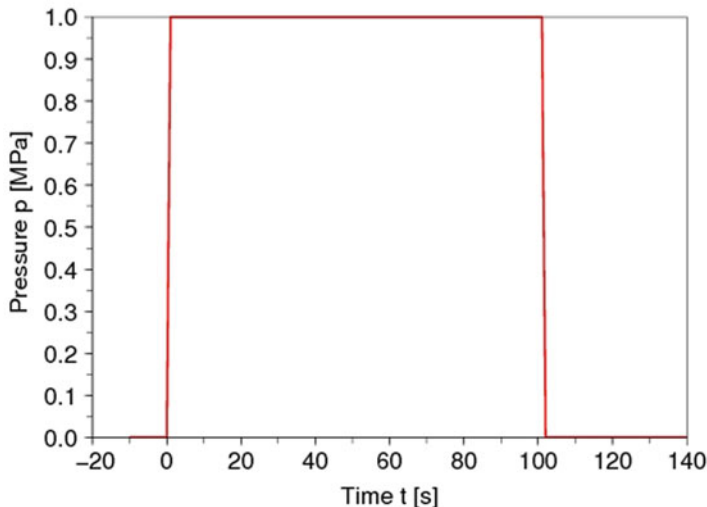


Fig. 4.84 Creep loading/unloading considered for static wheel (loading and unloading takes 1 s)

1 Hz. The position x of the center of the wheel print in function of time t is expressed as:

$$x = -20 \cdot \cos(2\pi t) \text{ (in cm)} \quad (4.45)$$

The load is supposed to be applied at $t = 0$, at the same time as the wheel starts moving.

Material Behaviour

The material is assumed to be isotropic linear viscoelastic. The Poisson's ratio is taken as a constant, equal to 0.35. To model the complex modulus a generalised Kelvin-Voigt model with 20 elements is considered (Fig. 4.75). The chosen parameter values are those of a typical surface layer bituminous mix in France (so called "Béton Bitumineux Semi-Grenu", BBSG). They are given in Table 4.17 for a temperature equal to 20°C. The parameters of the equivalent generalised Maxwell model are also given.

Furthermore, the time-temperature superposition principle is considered to be fulfilled. Then, the parameters of the Kelvin-Voigt or Maxwell model may be obtained for any temperature with the following equation:

$$\eta_i(T) = \eta_i(20^\circ\text{C}) \cdot a_T(T, 20^\circ\text{C})^Z \quad (4.46)$$

with a_T coefficient given by the WLF law, for $T_R = 20^\circ\text{C}$:

Table 4.17 Parameters of one dimensional Kelvin-Voigt and Maxwell models calibrated for a classical French BBSG (“Béton Bitumineux Semi-Grenu”)

Kelvin-Voigt model				Maxwell model		
<i>E0</i>	4.00E + 04			<i>E0</i>	6.00E + 01	
<i>E1</i>	2.75E + 06	$\eta1$	1.10E-06	<i>E1</i>	5.75E + 02	$\eta1$ 2.26E-10
<i>E2</i>	2.69E + 06	$\eta2$	1.07E-05	<i>E2</i>	5.70E + 02	$\eta2$ 2.23E-09
<i>E3</i>	2.11E + 06	$\eta3$	8.38E-05	<i>E3</i>	7.07E + 02	$\eta3$ 2.76E-08
<i>E4</i>	1.32E + 06	$\eta4$	5.27E-04	<i>E4</i>	1.08E + 03	$\eta4$ 4.16E-07
<i>E5</i>	8.14E + 05	$\eta5$	3.24E-03	<i>E5</i>	1.63E + 03	$\eta5$ 6.21E-06
<i>E6</i>	4.98E + 05	$\eta6$	1.98E-02	<i>E6</i>	2.39E + 03	$\eta6$ 8.89E-05
<i>E7</i>	3.02E + 05	$\eta7$	1.20E-01	<i>E7</i>	3.35E + 03	$\eta7$ 1.20E-03
<i>E8</i>	1.78E + 05	$\eta8$	7.09E-01	<i>E8</i>	4.41E + 03	$\eta8$ 1.50E-02
<i>E9</i>	1.00E + 05	$\eta9$	3.99E + 00	<i>E9</i>	5.43E + 03	$\eta9$ 1.71E-01
<i>E10</i>	5.12E + 04	$\eta10$	2.04E + 01	<i>E10</i>	6.17E + 03	$\eta10$ 1.72E + 00
<i>E11</i>	2.25E + 04	$\eta11$	8.96E + 01	<i>E11</i>	6.15E + 03	$\eta11$ 1.42E + 01
<i>E12</i>	8.21E + 03	$\eta12$	3.27E + 02	<i>E12</i>	4.58E + 03	$\eta12$ 7.95E + 01
<i>E13</i>	2.46E + 03	$\eta13$	9.78E + 02	<i>E13</i>	2.15E + 03	$\eta13$ 2.71E + 02
<i>E14</i>	5.15E + 02	$\eta14$	2.05E + 03	<i>E14</i>	6.13E + 02	$\eta14$ 5.90E + 02
<i>E15</i>	8.03E + 01	$\eta15$	3.20E + 03	<i>E15</i>	1.31E + 02	$\eta15$ 1.26E + 03
<i>E16</i>	7.42E + 02	$\eta16$	2.95E + 05	<i>E16</i>	4.62E + 00	$\eta16$ 1.70E + 03
<i>E17</i>	3.61E + 03	$\eta17$	1.43E + 07	<i>E17</i>	1.10E + 00	$\eta17$ 4.29E + 03
<i>E18</i>	1.15E + 05	$\eta18$	4.58E + 09	<i>E18</i>	8.31E-02	$\eta18$ 3.30E + 03
<i>E19</i>	3.12E + 06	$\eta19$	1.24E + 12	<i>E19</i>	8.88E-02	$\eta19$ 3.53E + 04

$$\log(a_T) = \frac{-C_1(T - T_R)}{C_2 + (T - T_R)} \quad \text{with } T \text{ in } ^\circ\text{C} \quad (4.47)$$

The two coefficients C_1 and C_2 are respectively equal to 30.5 and 243.6.

Calculation Results

The results are obtained at the centre of the slab ($x = y = 0$ cm). Two points are taken into account at two depths: point O_{10} at $z = 0.1$ m, which represents the top surface and point O_5 at $z = 0.05$ m, which represents the middle of the layer. These points are plotted in Figs. 4.83 and 4.85.

4.4.2.2 Results of the Finite Elements Analysis

The results presented in this section were obtained using the Finite Element code COMSOL. They were also confirmed by another team, which used an open source code CASTEM. In the two cases, the Kelvin-Voigt model was implemented in the code.

The mesh used in COMSOL code is presented in Fig. 4.85.

The results for static and rolling wheel are presented hereafter.

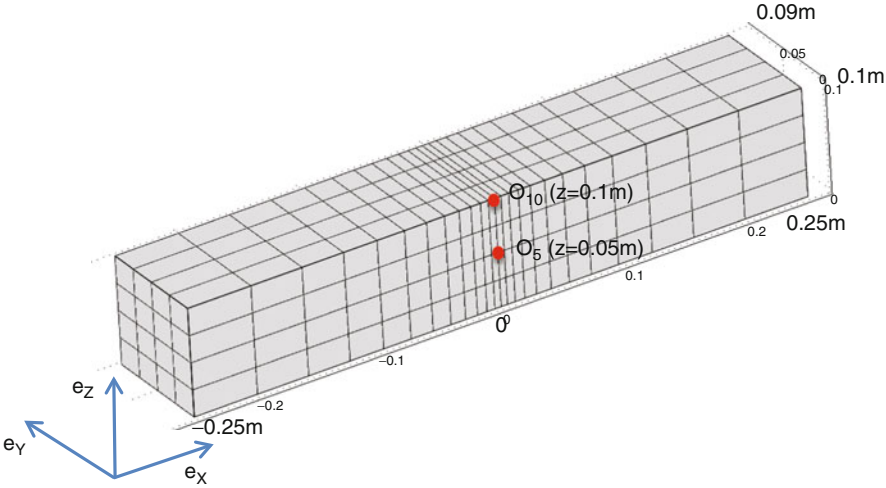


Fig. 4.85 Mesh of the modeled half-slab used for Finite Element analysis

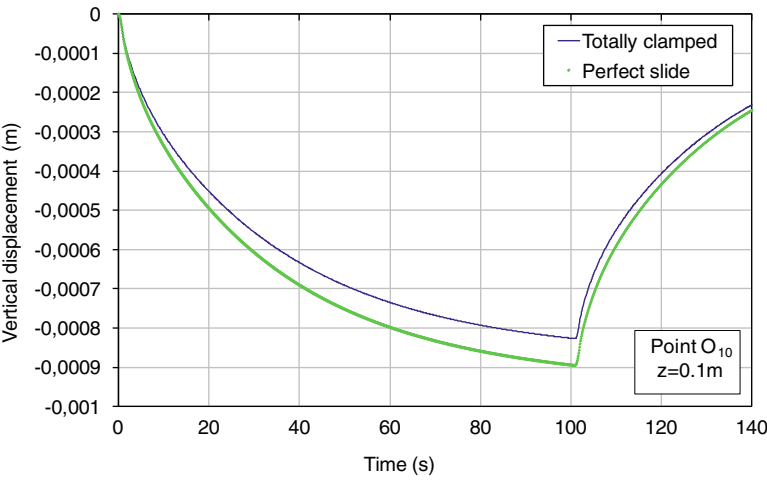


Fig. 4.86 Vertical displacement at the centre of the top surface of the slab (Point O_{10} in Fig. 4.85) under a static loading/unloading

Static Wheel, Creep Loading/Unloading, $T = 20^{\circ}\text{C}$

The first considered loading condition is a static wheel, applying a loading/unloading cycle on the slab. The calculation was performed with a temperature $T = 20^{\circ}\text{C}$ for the two boundary conditions previously detailed, namely totally clamped or perfect slide. The vertical displacement at the centre of the top surface of the slab (point O_{10} in Fig. 4.85) is presented Fig. 4.86. A small difference appears

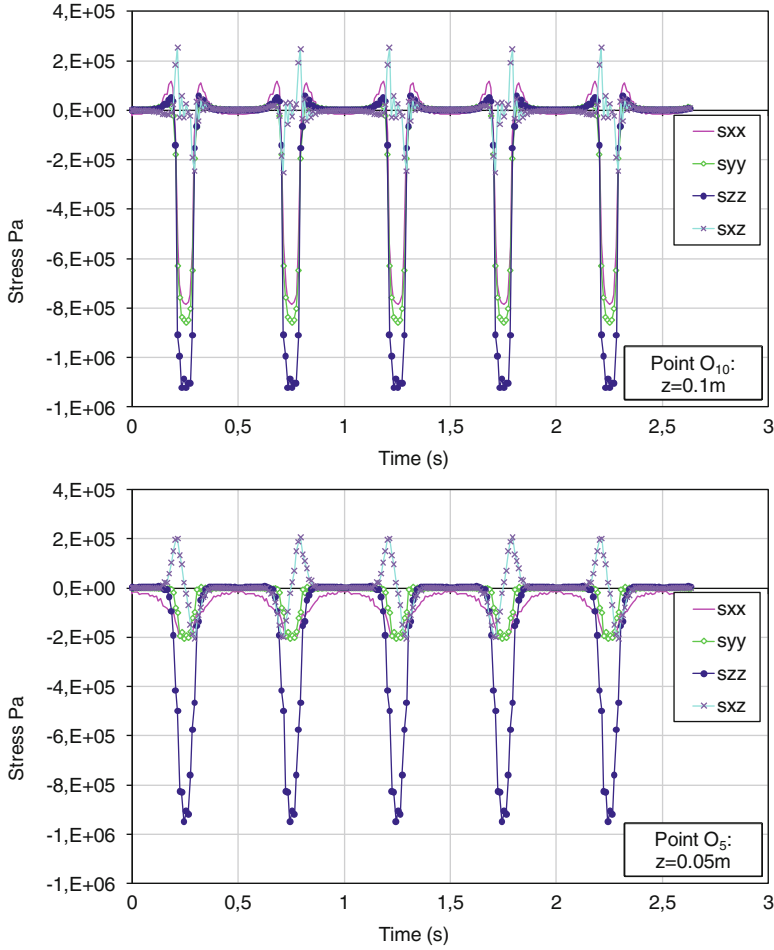


Fig. 4.87 Stress state at the centre of the slab under a rolling wheel, at the top surface (Point O₁₀ in Fig. 4.85) and at the mid-depth (Point O₅ in Fig. 4.85), whichever the boundary conditions and temperatures

between the two boundary conditions. As expected, the displacement obtained for the “perfect slide” condition is greater.

Moving Wheel

The second considered loading condition is a moving rectangular load, reproducing the rolling wheel of the French Wheel Tracking Test FWTT (see Fig. 4.21). Not only, both boundary conditions (“perfect slide” and “totally clamped” as explained earlier) were considered but also two temperatures $T = 20^{\circ}\text{C}$ and $T = 60^{\circ}\text{C}$.

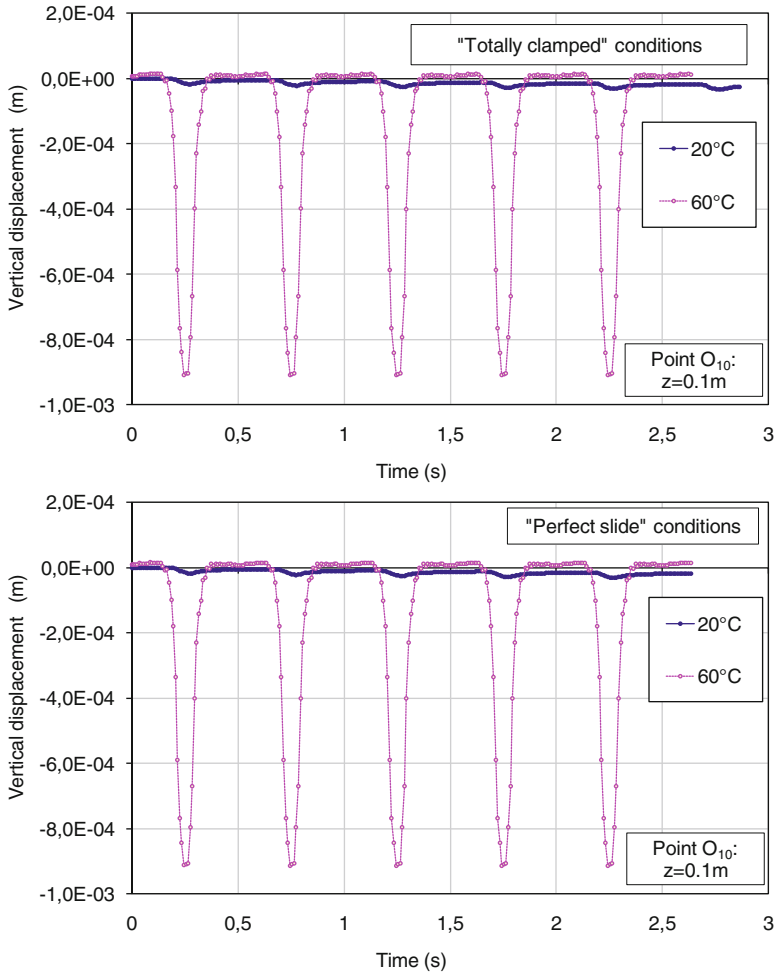


Fig. 4.88 Vertical displacement at the centre of the top surface of the slab (point O₁₀ in Fig. 4.85) under a rolling wheel, for two boundary conditions ("perfect slide" and "totally clamped") and two temperatures 20 and 60°C

First the stress state under the wheel, at the top surface and the mid-depth of the slab is presented in Fig. 4.87. It has to be noted that this stress state does not depend on the temperature or the boundary conditions. The Fig. 4.87 shows clearly the difference between the horizontal stress s_{xx} and s_{yy} at two depth in the slab.

Then the vertical displacement at the centre of the top surface of the slab is plotted in Fig. 4.88 for the first cycles. The difference between both boundary conditions is not noticeable, while the temperature shows a great influence. The amplitude of the displacement is much higher at 60°C than 20°C. Nevertheless, at 20°C, a small settling appears after the two first cycles, which is not the case at 60°C. A greater number of cycles was computed for temperature $T = 20^\circ\text{C}$.

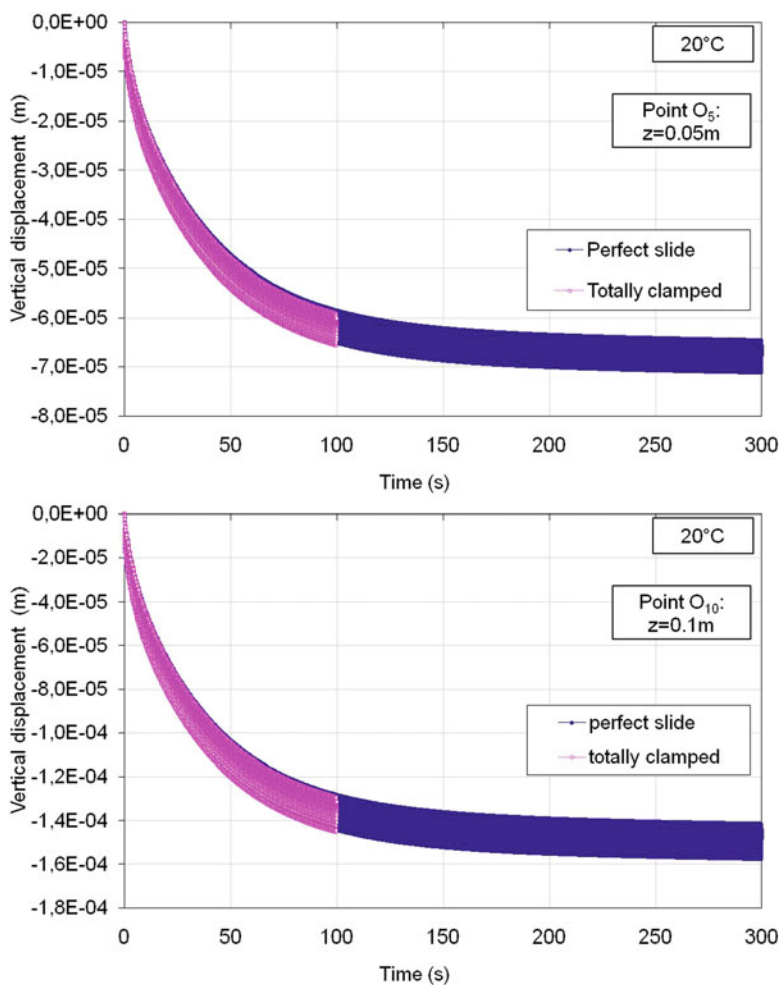


Fig. 4.89 Vertical displacement at the centre of the slab, at the *top* surface (Point O₁₀ in Fig. 4.85) and at the mid-depth (Point O₅ in Fig. 4.85), for two boundary conditions (“perfect slide” and “totally clamped” as explained in Sect. 4.2.1.2) and temperature 20°C

The obtained vertical displacement is plotted for both considered boundary conditions in Fig. 4.89. This figure shows the low influence of these boundary conditions on the vertical displacement.

These observed phenomena could be explained by the considered linear visco-elastic behaviour of the material. On one hand, the rigidity of the material is higher at lower temperatures, which induces lower displacement underneath the wheel. On the other hand, the viscous effects are less important at 60°C than 20°C and the material exhibits nearly an elastic behaviour, with a displacement null when the wheel has gone. At 20°C, the viscous effect induces delayed deformation and

vertical settling accumulates at each wheel passing. Nevertheless, this is not permanent deformation and it is expected that the vertical displacement becomes null if the loading wheel is removed.

4.4.2.3 Conclusions

The presented linear viscoelastic simulation should be considered as a first necessary step towards simulation of the French WTT. The influence of temperature is clearly shown. The difference between the two sets of boundary conditions (“totally clamped” and “perfect slide”) is not visible for this level of deformation. The difficulty to simulate a great number of cycles should also be emphasized.

In order to correctly take into account the behaviour of the material, more complex model introducing plastic non-linearity must be considered and implemented in FE software. It is the next step of this research.

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Chapter 5

Interlaboratory Tests on Pavement Performance Prediction and Evaluation – Rutting

Herald Piber, Manfred N. Partl, and Christiane Raab

Abstract The former RILEM TC 182 PEB organized 1997 an international interlaboratory test on pavement performance, prediction and evaluation. The aim was to predict the expected damages after 10 years on the basis of traffic and climatic data by testing samples that were cut from the newly constructed road. One section in Austria was monitored as planned until 2007. Thirteen laboratories from 12 countries participated in the study and provided performance predictions for both sections according to their own methods and standards. This chapter summarises the road condition after this long term performance observation and compares it to the rutting results from those laboratories which performed dynamic rutting tests on more than one pavement layer and based their prediction on their own test results and models without relying on national requirements.

Keywords Pavement • Long-term performance • Test section • Field measurement • Rutting • Traffic data

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5.1 Introduction

With increasing number of vehicles and axle loads on motorways the long term performance of asphalt pavements and its prediction are gaining more and more importance. In 1997 the former RILEM TC 182 PEB – followed by TC 206-ATB – organized an international interlaboratory test with laboratories participating from around the world. For this purpose, two test sections, one in a moderate climate (Portugal) and another one in a climate with high summer and low winter temperatures (Austria), were constructed. Test slabs were cut from the pavements and samples sent to different laboratories, which were asked to predict the damage for a 10 years period. The prediction was based on testing the distributed asphalt samples as well as the provided traffic and climate data. Each laboratory was asked to use its own methodology and test procedures. The results are presented in detail in Partl and Piber [11] as well as Piber and Partl [12].

Meanwhile the test sections were further monitored and eventually the predicted behavior was compared to the in situ performance. Since only one of the sections, the section in Villach (Carinthia), Austria, could be monitored over the whole observation period, the condition of this section after 10 years trafficking was compared to the predicted performance (Piber et al. [13]).

While all laboratories predicted very consistently single cracking, net cracking and surface defects, the results of the predicted rut depths varied in a wide range (Piber et al. [13]). Therefore, it was decided to conduct an additional in depth analysis of data from selected laboratories on the basis of the earlier reports (Partl and Piber [11]; Piber and Partl [12]; Piber et al. [13]), focusing on the behavior of rutting.

5.2 Pavement of the Test Section

The Austrian test section was constructed on the “A 10 Tauernautobahn”. This motorway provides the shortest connection over the Alps between the region of the northern part of the Adriatic Sea (Ljubljana – Slovenia, Trieste, Udine – Italy), the southern part of Austria (Carinthia) and the northern part of Austria (Salzburg) and Germany (Munich). Construction started in 1988 and was accomplished 1997. The structure of the test section is shown in Fig. 5.1.

5.3 Rutting Measurements and Predictions by the Laboratories

As indicated in Table 5.1, 13 laboratories from 12 countries participated in the study and received asphalt samples ($250 \times 550 \times 325$ mm) cut from the pavements for mechanical testing and evaluation.

All laboratories were asked to conduct their testing and evaluation based on their own national standards and methods. Their damage prediction for a 10 years period

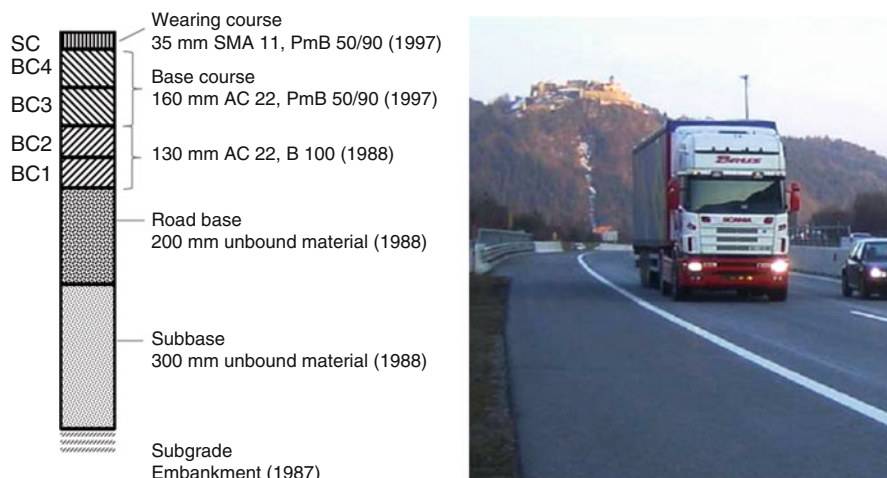


Fig. 5.1 Pavement structure of the test section (binder designation at time of construction)

Table 5.1 Participating laboratories and their reporters

Code	Laboratory	Reporter
AT	Bautechnik – Amt der Kärntner Landesregierung	Piber H.
BE	BRRC – Belgian Road Research Centre	Francken, L., Léonard, D. Vanelstraete, A.
CH	EMPA – Swiss Federal Laboratories for Material Science and Technology	Partl, M.N.
CZ	Technical University of Brno	Kudrna, J.
DE 1	Asphalt-Labor	Damm, K.W.
DE 2	BAM – Bundesanstalt für Materialforschung und -prüfung	Recknagel, Ch., Vater, E.J.
DK	DRI – Danish Road Institute	Bredahl Nielsen, C.
ES	CEDEX – Centro de Estudios de Carreteras	Baena – Rangel, J.M.
IT	University of Parma	Montepara, A.
PT	Consulpav	Sousa, J.M.B.
SE	VTI – Swedish National Road and Transport Research Institute	Said, S., Jansson, H.
ZA	CSIR – Transportek	Jooste, F.J., Vlok, M., Verhaeghe, B.M.J.A.,
US	University of California	Hung, D., Monismith, C.L.

had to be assigned to performance prediction categories A to D for the damage types rutting, single cracking, net cracking and surface defects as shown in Table 5.2 and reported in Partl and Piber [11] and Piber et al. [13].

According to the results in Table 5.2, damages such as single cracking, net cracking or surface defects were negligible and agreement between the different laboratory predictions was quite good. In contrast, rutting proved to be the most critical parameter for long term performance and the rutting prediction by all laboratories showed significant differences.

Six laboratories predicted the development of the rut depth from their own rutting test methods, while others based their prediction on the test results compared to the

Table 5.2 Interlaboratory results; labs per performance prediction category (A–D)

Type of damage											
Rutting				Single cracks (thermal cracking)		Net cracks (fatigue cracking)				Material loss and/or raveling	
mm				m/100 m		% of 100 m ²				% of 100 m ²	
<5	5–10	10–20	≥20	<4	≥4	<5	5–10	10–20	20	<2	≥2
A	B	C	D	A	B	A	B	C	D	A	B
Number of laboratories corresponding to the performance prediction categories											
5	3	4	0	4	0	9	0	0	0	2	0

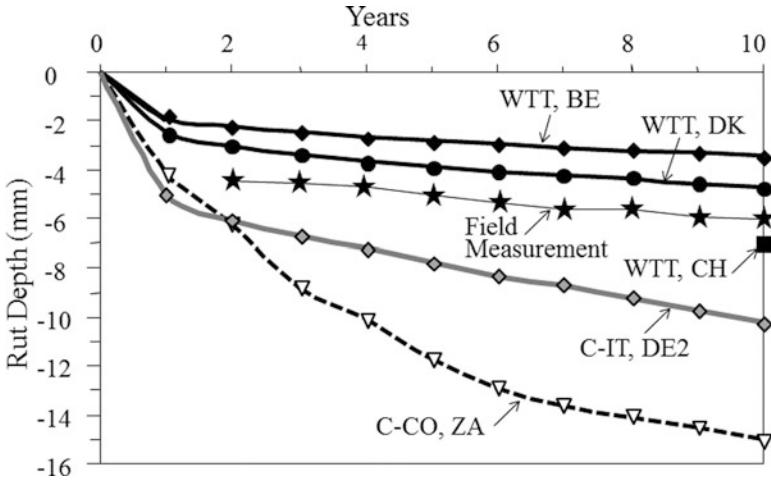


Fig. 5.2 Calculated rutting from different dynamic lab tests in comparison with field measurement

requirement of their national regulation or standard (Piber et al. [13]). This additional analysis focuses on those five laboratories that based their prediction on the following dynamic tests on more than one layer without relying on national requirements: Cyclic compression test C-CO (ZA, DK), cyclic indirect tensile test C-IT (DE2) wheel tracking tests WTT (BE, CH, DK). Since all laboratories that performed rutting tests tested the different layers either separately or as multilayer system with considerably differing test equipment and conditions, only the rutting predictions by the labs but not the individual test results could be compared in a direct way.

In Fig. 5.2 the rutting prediction results for all laboratories are shown and compared with the field measurements. The in situ measured rut depth after 10 years monitoring was 6.1 mm (± 2.4 mm). Comparing this result with the laboratory prediction shows that the three laboratories with the wheel tracking test WTT came very close to this value. All three laboratories used a testing machine with a pneumatic tire (400 $\times$ 80 mm). However, the sample size varied. Some laboratories tested each course separately others tested the courses together.

Table 5.3 Volumetric values of the wearing course and the top layer of the base course

Course	Property/Laboratory	AT	BE
SC	Binder content (% by mass)	6.6	6.18
	Maximum density (kg/m^3)	2,603	2,638
	Bulk density (kg/m^3)	2,541	2,581
	Binder content (% by volume)	16.44	15.64
	Mineral content (% by volume)	81.18	82.20
	Air void content (% by volume)	2.38	2.16
BC 4	Binder content (% by mass)	4.6	4.23
	Maximum density (kg/m^3)	2,604	2,643
	Bulk density (kg/m^3)	2,528	2,512
	Binder content (% by volume)	11.40	10.42
	Mineral content (% by volume)	85.68	84.63
	Air void content (% by volume)	2.92	4.96

Table 5.4 Binder properties

Course	Property//Laboratory	AT	BE	CZ	DK	ZA
SC	Penetration 25°C (0.1 mm)	40	39	41	48	39
	Softening point (°C)	60.1	61.5	61.4	60.5	65.0
BC 4	Penetration 25°C (0.1 mm)	35	26	35	33	34
	Softening point (°C)	61.4	64.5	62.0	62.8	65.0

5.3.1 Asphalt Mixture and Binder

The quality of the wearing or surface course SC and the top layer of the base course BC 4 is the key to prevent rutting. The investigation and evaluation is concentrated on this topic. Mixture and binder characteristics of SC and BC4 as determined by different laboratories are given in Tables 5.3 and 5.4.

5.3.2 Stiffness Modulus

5.3.2.1 Laboratory: BE

Measurements of the stiffness modulus (E^*) and phase angle (φ) were performed for all layers by means of two-point bending tests using trapezoidal specimens. The following temperatures and frequencies were selected: -20 , -10 , 0 , 10 , 20 , 30°C and 1 , 3 , 10 , 30 Hz. From these measurements the master curves were calculated according to Eq. (5.1) with the parameters and coefficients for E^* and φ as reported in Table 5.5 (Francken and Vanelstraete [7]).

Table 5.5 Lab BE, parameters determining the master curve of E^* and φ for the wearing course SC and top layer of the base course BC 4

Course		SC		BC 4	
Parameter		E^*	φ	E^*	φ
δH_E (kcal/mol)	δH_φ (kcal/mol)	49.0	84.5	47.0	77.5
Coefficients					
a_0	b_0	3.6138	24.3383	3.8226	19.3283
a_1	b_1	0.3314	−5.183	0.2842	−4.8599
a_2	b_2	−0.03468	0.0799	−0.040	0.4173
a_3	b_3	−0.006525	0.1500	−0.00183	0.05222
a_4	b_4	0.002201	−0.0300	0.00133	−0.02334
a_5	b_5	−0.000215	0.002538	−0.000149	0.00295
a_6	b_6	0.00000708	−0.000082	0.0000054	−0.000125
Glassy modulus (MPa)	–	33,468	–	33,434	–

$$\begin{aligned} \log(E^*) &= a_0 + a_1 \times x + a_2 \times x^2 + a_3 \times x^3 + a_4 \times x^4 + a_5 \times x^5 + a_6 \times x^6 \\ \varphi &= b_0 + b_1 \times x + b_2 \times x^2 + b_3 \times x^3 + b_4 \times x^4 + b_5 \times x^5 + b_6 \times x^6 \\ x &= \log(\alpha_T) + \log(f) \end{aligned} \tag{5.1}$$

where:

- E^* = stiffness modulus
- x_E, φ = reduced frequency depending whether E^* or φ
- a, b = coefficients
- $\alpha_T = \exp(\delta H/R (1/T - 1/T_S))$; δH either δH_E or δH_φ depending whether E^* or φ
- f = frequency
- T = temperature ($^{\circ}\text{K}$)
- T_S = reference temperature (here $288^{\circ}\text{K} = 15^{\circ}\text{C}$)
- R = universal gas constant ($0.00198 \text{ kcal/mol/K}$)

5.3.3 Wheel Tracking Test (WTT)

5.3.3.1 Test Device

The WTT using the large device with pneumatic tire was carried out by three laboratories. Configuration of the various devices and test conditions are presented in Table 5.6 using the notations given in Fig. 5.3.

5.3.3.2 Test Results

Laboratory: BE

Measurements of the permanent deformation were performed with the LCPC rutting tester at 35 and 45 $^{\circ}\text{C}$ on the SMA surface course (SC) and on the second

Table 5.6 Labs BE, CH, DK, configurations of WTT, specimen size and test temperatures

Characteristics		Laboratory		
		BE	CH	DK
Pneumatic tire type and dimension (mm)		Without profile 400 × 80	Without profile 400 × 80	With profile 400 × 80
Tire pressure p (kPa)		600	600	600
Load L (kN)		5	5	5
f (Hz)		1	1	0, 6
l _{WT} (mm)		410	410	410
Test temperature (°C)		35, 45	38, 60	20, 25, 40, 50
Specimen dimensions (mm)	l	500	480	550
	w	180	160	250
	h	30–100	40–100	220–250
	d	–	–	–
Results		Deformation curve		
		–	Secant modulus	–

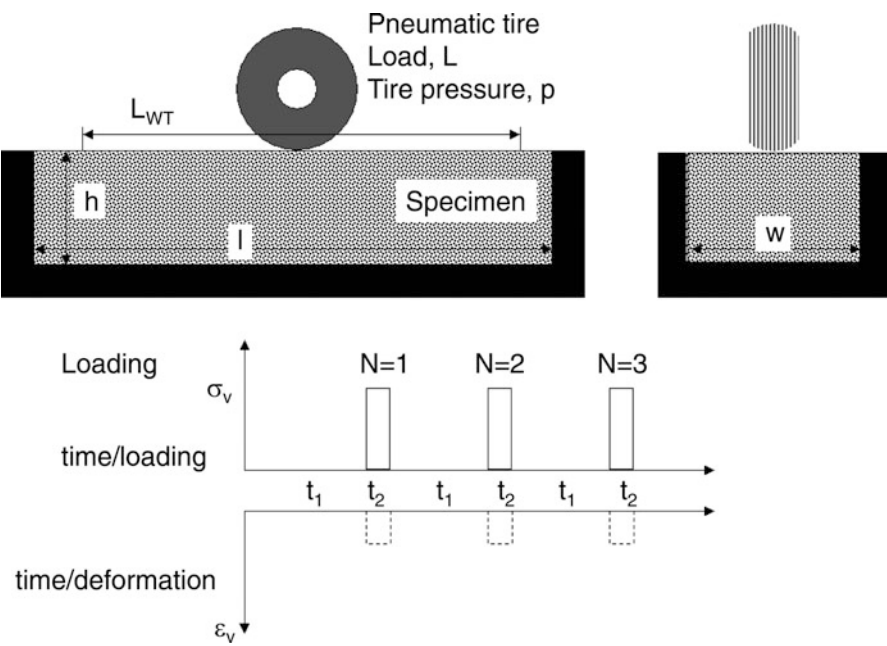


Fig. 5.3 Wheel tracking test with pneumatic tire

layer BC 4. The Belgian test method CME 54.13 [2] was followed, which implies that the ruts are determined after 10,000, 20,000, 30,000, 50,000, 70,000 and 100,000 loading cycles. According to this procedure, the ruts are determined with respect to the “lips” of the transversal profile. Two specimens are tested per temperature. The results are listed in Table 5.7. The permanent relative deformation

Table 5.7 Lab BE, results of the rutting tests on the SC and BC 4 layers at 35 and 45°C

Course	SC				BC 4			
Number of the specimen:	1	2	3	4	1	2	3	4
Temperature (°C)	35	35	45	45	35	35	45	45
Density (g/cm ³) (geom.)	2.537	2.514	2.49	2.532	2.49	2.494	2.518	2.507
Sample thickness (mm)	32.4	31.9	30.6	33.5	60.1	62.8	62.3	59.8
Relative deformation (%) after:								
10,000 cycles	5.99	6.68	6.57	5.70	2.50	2.32	4.86	3.63
20,000 cycles	6.64	7.52	7.45	6.30	2.93	2.68	5.65	
30,000 cycles	6.88	7.90	8.20	6.72	3.31	2.96	6.47	4.62
50,000 cycles	7.78	9.06	8.95	7.13	4.03	3.58	7.74	5.28
70,000 cycles	8.12	9.56	9.74	7.55	4.38	3.87	8.20	5.64
100,000 cycles	9.07	10.63	10.36	7.88	4.91	4.32	8.73	6.05
Regression parameters:								
a	0.176	0.200	0.200	0.141	0.301	0.276	0.268	0.225
$\varepsilon_{p(10,000)}$ (%)	5.99	6.68	6.54	5.72	2.44	2.26	4.83	3.63
mean a	0.19		0.17		0.29		0.25	
mean $\varepsilon_{p(10,000)}$ (%)	6.3 ± 0.4		6.1 ± 0.4		2.4		4.2	

ε_p in % of the original specimen thickness can be described by a straight line represented on a bi-logarithmic curve. Therefore, the following power function is applicable:

$$\varepsilon_p = \varepsilon_{p(10,000)} \times N_{(10,000)}^a \quad (5.2)$$

where:

$\varepsilon_{p(10,000)}$ = relative permanent deformation after 10,000 cycles

$N_{(10,000)}$ = number of cycles divided by 10,000 cycles

a = parameter

Laboratory: CH

The LCPC-WTT test was performed with a pneumatic wheel tracking device made by MLPC France and developed by LCPC based on the French Standard NF P 98-253-1 [9].

The loading system is mounted in a temperature chamber. It consists of two tables each carrying a steel mold for a specimen 100 × 180 × 500 mm which is loaded by treadles pneumatic rubber tire (Trelleborg T-522 BV) of 400 mm diameter and an active width of about 80 mm (±50 mm). Tables and specimens are pressed pneumatically with 5 kN (±50 N) desired load against the tires which are guided back and forth along linear guiding rails over a path of 410 mm (±5 mm) length at a frequency of 1 Hz. Testing temperature is controlled with thermocouples measuring the temperature inside the specimens.

Rut depth is determined with LVDT's in 15 positions. The test is interrupted at the desired number of load cycles, typically after 0, 30, 100, 300, 1,000, 3,000, 10,000, 30,000. The number of load cycles (1 cycle = 2 wheel passes) is counted

Table 5.8 Lab CH, results of the rutting test on SC + BC 4 and BC 4 + BC 3 courses

Course	SC + BC4		BC4 + BC3	
Number of the specimen	2	1	3	4
Temperature (°C)	38	60	38	60
Sample thickness (mm)	SC = 34	SC = 34	BC4 = 64	BC4 = 64
	BC4 = 66	BC4 = 66	BC3 = 35	BC3 = 35
	(Σ = 100)	(Σ = 100)	(Σ = 100)	(Σ = 100)
Relative deformation (%) after:				
30 cycles	0.244	0.871	0.187	0.769
100 cycles	0.429	1.383	0.324	1.142
300 cycles	0.615	2.030	0.462	1.452
1,000 cycles	0.751	2.855	0.596	1.847
3,000 cycles	1.126	4.627	0.779	2.738
10,000 cycles	1.381	13.563	1.035	4.048
21,758 cycles		16.991		
30,000 cycles	1.593		1.419	5.037
Regression parameters:				
k	0.119	0.158	0.085	0.307
q	0.266	0.456	0.277	0.273

and controlled by a mechanical counter. Before conducting the test, the specimen is pre-loaded with 1,000 cycles at a temperature between 15 and 25°C.

From the 550 × 250 mm slabs of variable height, specimens of a surface area of 480 × 160 mm (±1 mm) were cut and embedded in the 500 × 180 steel molds of the WTT by using cement mortar.

The rutting results were approximated by a power function

$$\varepsilon_p(T) = k(T) \times N^{q(T)} \quad (5.3)$$

where

ε_p = relative deformation in %.

N = number of cycles

k, q = regression parameters in Table 5.8.

All data are mean values of the 15 measurement points of only one sample. At 60°C, the samples from Villach suffered heavy rutting because these mixes were designed for hot climate but for an alpine climate situation. Note that the SC + BC 4 samples from Villach failed at 60°C only after 21,758 cycles.

Laboratory: DK

The Danish Asphalt Rut Tester (DART) is a stationary accelerated load testing facility with a full-size standard truck tire, 5 kN load and a linear speed of 5 km/h. It has been calibrated for large slabs (1,500 by 1,200 mm) with in-field rut depth measurements of Danish motorway pavements (Nielsen [10]).

As an option, DART can be mounted with a small loading wheel fitted with a 400×8 pneumatic tire. The loading wheel is positioned at one end of the slab and moves 410 mm with constant load at a frequency of 0.6 Hz. The load is 5.0 kN and the tire pressure is 600 kPa. This small-scale option was selected here due to the size of the slabs received for the RILEM tests. However, since no calibration with in field measurements was available for the small-scale operation mode, a purely empirical adoption to the large scale test model had to be made, for this investigation. The general test procedure for DART in this small-scale mode is given in the test procedure of the Danish Road Institute [4].

Wheel tracking tests were performed on one slab. The maximum thickness of slabs to be tested in wheel-tracking at the Danish Road Institute is 250 mm. It was therefore decided only to test the layers above BC1, i.e. above the geotextile that had been inserted locally during pavement construction for sampling purposes. The slab was carefully separated with hammer and chisel in the lower base course along the geotextile. A weak Portland cement mortar was used to level the underneath of the slab out and to ensure a parallel, plane support during testing. The slab was turned upside down and cast with a 40 mm thick weak Portland cement mortar (one part Portland cement and seven parts 0/8 mm gravel) along the vertical sides of the slab. The slab was then placed in the center of the test mold with overall dimensions of 1,200 by 1,500 mm. The surplus space was filled with 0/8 mm well compacted gravel to a level 50 mm below the surface of the slab. The upper 50 mm in the test mold was then filled with a weak Portland cement mortar flush with the surface of the slab.

During wheel tracking tests, the slabs were conditioned at a constant temperature of 40°C at the surface and 20°C at the bottom of the slabs. These temperatures were controlled within $\pm 1^\circ\text{C}$ and measured four times per hour.

Temperatures within the slabs were not measured during the testing, but slabs were conditioned at least 24 h prior to testing. After 45,000 loads, the test was stopped.

For any specified number of loads, five cross profiles and three longitudinal profiles were measured with a laser profilometer. Cross profiles were measured at the cross center line of the slab and ± 50 mm and ± 100 mm away from this line. The longitudinal profiles were measured at the longitudinal center line of the slab and at a distance of ± 5 mm from this line. Vertical distances from a horizontal reference plane to the surface of the slab were measured. Each profile was measured with a 1 mm horizontal spacing between each measurement point. Rut depth was calculated as average value over the 50 mm wide middle zone of all cross profiles and the 200 mm wide middle zone of all longitudinal profiles.

After wheel-tracking tests, three 100 mm diameter cores were drilled from each slab to determine material composition and air void content.

The rut depth (P_N) after N loads was estimated by the power function:

$$P_N = a \cdot N^b \quad (5.4)$$

Table 5.9 Lab DK, results of DART wheel tracking test

Course	SC + BC4 + BC 3 + BC 2
Number of the specimen	1
Temperature (°C)	40 on the surface 20 on the bottom
Sample thickness (mm)	250
Regression parameters:	
a	0.0981
b	0.2756
R ²	0.871
Deformation (mm) after:	
20,000 cycles	1.5
40,000 cycles	1.8
Increase	0.3

By applying linear regression methods, the constants a and b were derived from the experimental data. To obtain a better fit for large numbers of loads, only data for 10,000 loads and more are used to derive the constants. The constants are given in Table 5.9. The power function is used to calculate rut depth at 20,000 and 40,000 loads. Furthermore, as a measure of the rutting rate, the increase in rut depth between 20,000 and 40,000 loads is calculated.

5.3.4 Dynamic Creep Test

5.3.4.1 Laboratory: DK

The dynamic uniaxial unconfined compression creep test was performed according to a Swedish test procedure FAS 468–97 [5], using the Nottingham Asphalt Tester (NAT). The test procedure is similar to BS DD 185 performed on 150 mm diameter cores with a 100 mm loading disc. All specimens were cored from the pavement slabs and tested at 40°C. In Table 5.10 the total creep after 3,600 loads and the creep rate f^* , i.e. the overall slope of the relative deformation curve between 2,500 and 3,600 loads, are given.

5.3.4.2 Laboratory: DE2

The dynamic indirect tension creep test was conducted on specimens with a diameter of 200 mm and a height of 250 mm that were cored in the direction horizontal to the pavement thus comprising along the vertical diameter the layers SC + B4 + B3. Loading was simulating the trucks at a speed of 80 km/h that were driving in a distance of 40 m. This led to the following repeated loading sequences consisting of a 0.02 s haversine with a load amplitude of 6.3 kN followed by a rest period of 0.2 s between steering and rear axle before the second 6.3 kN haversine

Table 5.10 Lab DK, dynamic creep mean values with standard deviation of 5 specimens

Course	SC + BC 4 + BC 3
Diameter and thickness of one core (mm)	D = 150, h = 200
Test temperature (°C)	40
Total relative deformation after 3,600 loads (mm/m)	5.5 ± 0.9
Creep rate f^* between 2,500 and 3,600 loads (mm/m/l)	0.24 ± 0.04

Table 5.11 Lab DE2, deformations due to a 10 year simulated traffic loading (horizontal coring)

Course		SC + BC 4 + BC 3	
Diameter and thickness of one core (mm)		D = 200, h = 250	
Test temperature (°C)	Estimated number of trucks in 10 years, including 3% increase	Testing time (h)	Deformation (mm)
44	61,034	35.3	4.11
40	85,754	49.5	4.97
34	610,844	352.9	1.67
35	964,446	557.2	0.31

load impulse of 0.2 s is applied that triggers the second rest period of 1.8 s representing the time to the next truck. Vertical and horizontal deformations are measured with LVDT's. The temperature regime of the Austrian test section over 10 years for a 3% traffic increase was simulated in the test by conducting the tests at four different temperatures 25, 34, 40 and 44°C (Table 5.11). The testing time at a certain temperature corresponds to the number of trucks that is expected at this temperature over 10 years.

5.3.5 Axial Loading Slab Test

5.3.5.1 Laboratory: ZA

The basic configuration of the Axial Loading Slab (ALS) test is illustrated schematically in Fig. 5.4. The slab is placed in a steel mould on a synthetic support of known stiffness. Three types of synthetic materials (representing low, medium and high stiffness supports) can be selected to approximate the support conditions for the planned asphalt layer.

This test is normally performed on slabs 380 by 340 mm that are constrained in a metal frame. The slabs available were smaller than the frame in which the sample needs to be constrained. Therefore, a standard SMA mixture was compacted around the test specimen to provide a realistic simulation of the lateral confinement afforded by the mix under normal circumstances.

The loading platen has a diameter of 100 mm. For this study, the SMA and the asphalt base layer were tested as one sub-layer system providing an overall rut depth for the upper part of the pavement system. The test is performed at 30, 40 and 50°C and with 700 kPa nominal stress. At each temperature level, 10,000 load

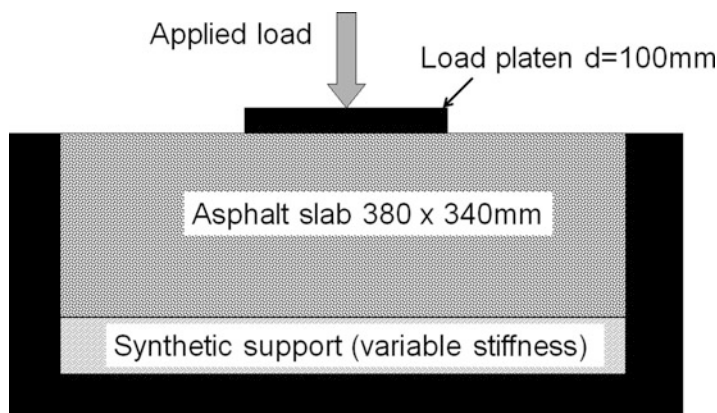


Fig. 5.4 Schematic representation of the ALS Test

repetitions are applied and the rut increment is monitored. The load is applied in a haversine wave shape. The load duration is determined on the basis of the expected heavy vehicle speed. For applications such as climbing lanes and intersections, where heavy vehicle speeds are often below 30 km/h, a load duration of 0.5 s is used, followed by a rest period of 0.5 s. For applications with typical heavy vehicle speeds above 40 km/h, a load duration of 0.2 s is used, followed by a 0.8 s rest period. For this analysis, a haversine wave with a duration of 0.2 s and a rest period of 0.8 s was used.

Test results were analyzed using an equation that relates rut increments with temperature, applied stress and number of load repetitions. The model requires detailed information on the axle load distribution. Since this information was not available for the Austrian test section, rut prediction in this case is limited to an explorative and not an accurate rut prediction.

The link between the test results of the ALS test (Table 5.12), the temperature conditions and the traffic data gives following equation:

$$\log(RI) = a + b \cdot \log(T) + c \cdot \log(P) + d \cdot \log(N) + e \quad (5.5)$$

where:

RI = Rut increment in mm

T = Temperature of the asphalt course in °C

P = Vertical stress in kPa

N = Number of axle repetitions

a, b, c, d, e = coefficients (Table 5.13)

Once a function relating rut increment to temperature, stress and load repetitions has been derived, it can be used to predict and compare rutting performance of the tested mix for different design situations. Such a fingerprint function can, for example, be used to evaluate the sensitivity of the mix to high temperatures and

Table 5.12 Lab ZA, test results of ALS

Course	SC + BC 4	BC 3	BC 2	BC 1
Test temperature (°C)	30	40	50	
Deformation (mm) after				
2,000 cycles	0.2	0.48	0.89	
4,000 cycles	0.27	0.63	1.21	
6,000 cycles	0.33	0.72	1.42	
8,000 cycles	0.37	0.77	1.52	
10,000 cycles	0.41	0.86	1.68	

Table 5.13 Lab ZA, regression results of ALS

a	b	c	d	e	R ²
−6.945	2.641	0.133	−0.541	0.095	0.90

overloading. It can also be used in models for predicting seasonal temperature variations and evaluating traffic loading in a rational manner. One such stochastic analysis model is the Probabilistic Rut Analysis System (PRORAS). It provides a means of performing a sophisticated evaluation of the mix for a reasonable estimate of rut depth development over the design life of the mix.

5.4 Field Measurements and Data Collection by the Road Operator

The task of the road operator of the test section was counting the traffic, measuring the temperatures of the air and in various depths of the asphalt pavement, observing the surface characteristics and measuring the rut depths.

The A 10 Tauern motorway has two separate carriageways with two lanes in each direction. The width of each lane is 3.75 m. The test section is situated between km 178.9 and 179.0 on the right lane of the carriageway direction “Salzburg”. The radius of a left turn is 2.500 m, the longitudinal gradient is 0.5% and the cross fall 3.4%. In the area of the test section the pavement lies on an embankment of about 8 m. There is a speed restriction of 100 km/h and a ban of passing for trucks.

Rut depths in the right and left wheel track were measured in ten cross profiles in a distance of 10 m (Fig. 5.5). The samples delivered to the laboratories were taken between profiles 5 and 6. The temperature measuring device was situated between profiles 5 and 6.

5.4.1 Traffic Census

The traffic census was done automatically. The number of vehicles and heavy vehicles for both directions is listed in Table 5.14 and the distribution of the average monthly daily heavy traffic (AMDHT) on the right lane of the direction to Salzburg is shown in Fig. 5.6.

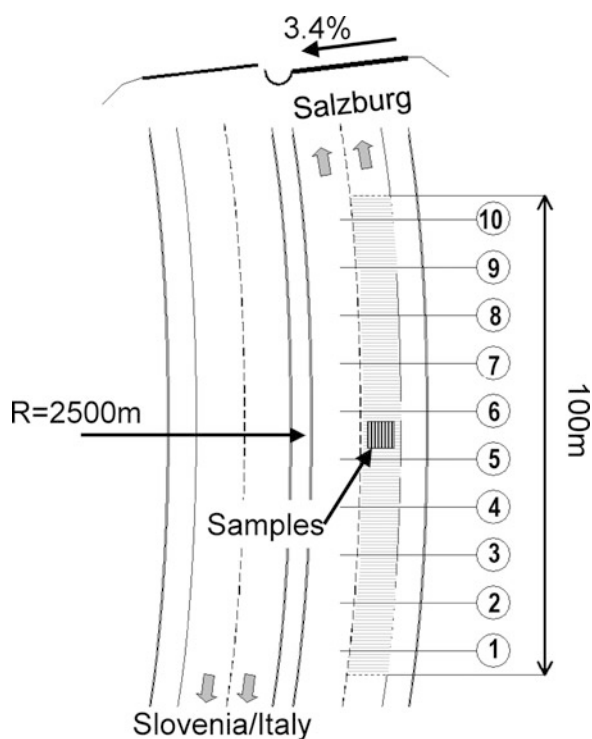


Fig. 5.5 Situation on the test section on the “A 10 Tauern Autobahn” (Villach, Carinthia, Austria)

Table 5.14 “A 10, Tauern Autobahn”, km 179.0, result of automatic traffic census

Year	Number of		Daily average	
	All vehicles	Heavy vehicles	AADT ^a	AADHT ^b
1998	5,108,338	1,164,608	13,995	3,191
1999	4,891,812 ^c	846,650 ^c	13,402	2,320
2000	5,524,842	1,046,217	15,137	2,866
2001	5,775,761	1,087,062	15,824	2,978
2002	5,932,512	1,151,148	16,253	3,154
2003	5,958,461	1,199,201	16,325	3,285
2004	4,948,441 ^d	902,571 ^d	13,557	2,473
2005	5,551,610	1,171,227	15,210	3,209
2006	5,379,715	1,093,847	14,739	2,997
Total/Average	Tot: 49,071,492	9,662,531	Avg: 14,938	2,941

^aAADT = Average annual daily traffic

^bAADHT = Average annual daily heavy traffic

^cFire accident in Tauern tunnel, tunnel closed for month in both directions

^dRehabilitation in Oswaldiberg tunnel – partly diversion of traffic on federal roads

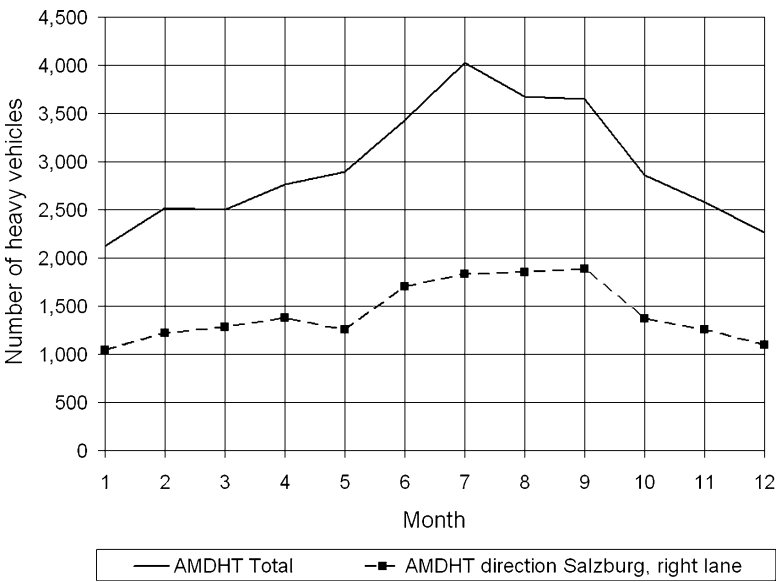


Fig. 5.6 Distribution of the average monthly heavy traffic – total and on the test lane

Table 5.15 Factor k for calculating the hourly heavy traffic on motorways

	Day	Evening	Night
Time:	6 a.m.–7 p.m.	7 p.m.–10 p.m.	10 p.m.–6 a.m.
Factor k	0.049	0.044	0.029

In summertime the number of heavy vehicles was above the average annual traffic. Therefore, it was necessary to calculate the heavy vehicles per hour (HHT) by using the following equation:

$$\text{HHT} = k \times \text{AMDHT} \tag{5.6}$$

where:

- HHT = hourly heavy vehicles
- k = factor (Table 5.15)
- AMDHT = average monthly heavy traffic

5.4.2 Heavy Traffic Characteristics

The composition of traffic in terms of different heavy vehicle types was counted yearly according to Fig. 5.7. The results are listed in Table 5.16.

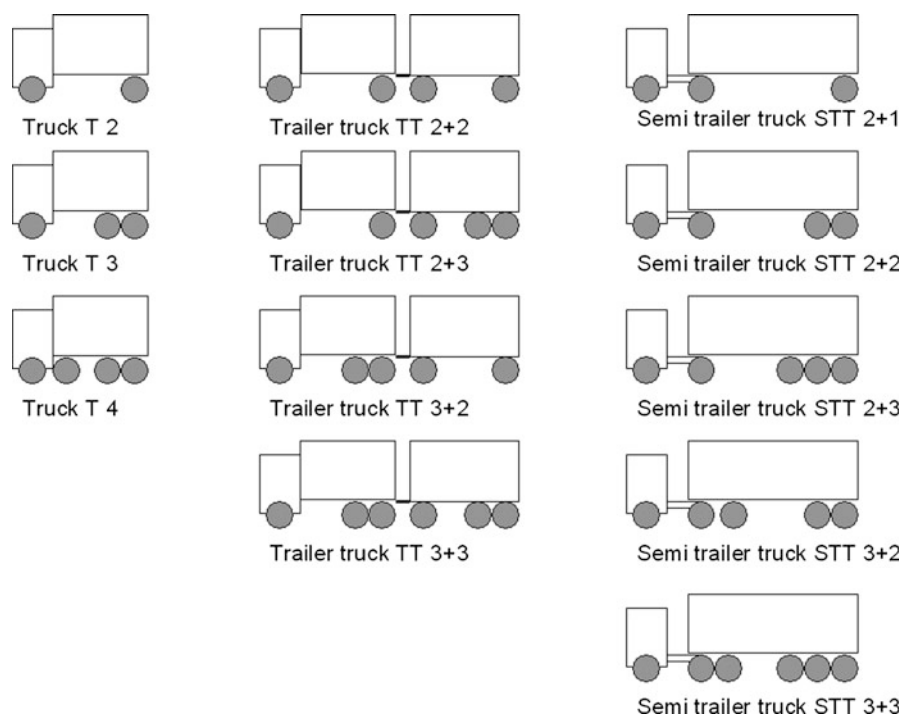


Fig. 5.7 Types of heavy trucks

5.4.3 Axles and Wheel Loads

The numbers of axles corresponding to Table 5.16 are listed in Table 5.17.

The axle load distribution was provided by the Institute of road construction of the Technical University of Vienna (ISTU Wien). In the frame of their research work, investigations of axle load distributions were carried out on Austrians motorways. The results are shown in Table 5.18.

The distribution of the wheel loads is calculated from the values of Tables 5.16 and 5.18 and presented in Table 5.19.

5.4.4 Tire Type and Tire Pressure

The type of tires and the tire pressures were not investigated during the period of observation. However, FEHRL investigated this topic in their research program COST 334 [3]. The report gives an overview on the distribution of tire types based on the investigation of 4,209 tires in total. The results are summarized in Table 5.20.

Table 5.16 Types of heavy trucks, gross vehicle weight and proportion in percent

Type	Weight (t)	1998	1999	2000	2001	2002
T2	<18	16.0	15.5	15.3	12.1	12.9
T3	<25	4.8	4.9	4.7	4.1	4.6
T4	<32	0.1	0.8	0.3	1.4	0.9
TT2 + 2	<36	7.2	7.8	6.1	6.7	7.4
TT2 + 3	<38	9.1	7.8	6.5	5.5	3.9
TT3 + 2	<38	10.0	12.2	12.8	10.8	12.1
TT3 + 3	<38	0.9	0.8	0.9	1.1	0.8
STT2 + 1	<36	0.3	0.5	0.5	0.7	0.4
STT2 + 2	<38	0.6	1.5	1.5	1.0	1.5
STT2 + 3	<38	50.3	47.5	50.2	55.3	53.7
STT3 + 2	<38	0.4	0.5	0.9	0.4	0.9
STT3 + 3	<38	0.3	0.3	0.2	0.8	0.8
Type	Weight (t)	2003	2004	2005	2006	Av.
T2	<18	10.6	12.9	11.1	12.9	13.3
T3	<25	5.0	3.4	5.6	3.9	4.6
T4	<32	0.2	0.1	1.8	0.3	0.7
TT2 + 2	<36	8.0	7.2	8.8	7.6	4.7
TT2 + 3	<38	5.2	4.2	3.6	5.1	5.7
TT3 + 2	<38	10.0	12.2	11.9	11.7	11.5
TT3 + 3	<38	1.3	0.1	0.8	0.6	0.8
STT2 + 1	<36	0.5	0.4	1.4	1.1	0.6
STT2 + 2	<38	1.1	0.6	0.9	1.5	1.1
STT2 + 3	<38	56.8	57.4	52.8	53.3	53.0
STT3 + 2	<38	0.5	1.1	1.1	1.6	0.7
STT3 + 3	<38	0.9	0.2	0.1	0.4	0.4

Table 5.17 Number of axles

Year	Number of axles	Axles per truck	Year	Number of axles	Axles per truck
1998	2,454,832	4.352	2003	2,726,646	4.502
1999	1,773,873	4.336	2004	1,934,331	4.458
2000	2,200,917	4.369	2005	2,457,526	4.421
2001	2,350,868	4.467	2006	2,370,605	4.429
2002	2,522,159	4.433	Total	20,791,757	4.421

Eliminating the unknown tires and distributing the known tire types (69%) to the axles and heavy vehicle categories truck and tractor as well as trailer and semi trailer, the relative tire type distribution in percentage of each column in Table 5.21 was found and used for further analyzing the heavy traffic loading of the A 10 test section.

Contact area is an important factor for pavement design. Inputs to mechanistic design models require the definition of the shape of the loaded area, which is generally assumed to be circular or elliptical. For present-day radial tires, the footprint is rather rectangular when tire load and inflation pressure are well

Table 5.18 Axle load distribution and heavy vehicle type in percent on the “A 10 Tauern Autobahn”

Axle load (t)	T2	T3	T4	TT 2 + 2	TT 2 + 3	TT 3 + 2
<1	0.0	0.9	1.6	0.0	0.0	0.0
1–2	0.0	3.5	3.5	4.3	0.5	0.3
2–3	0.8	2.4	9.1	2.6	4.2	1.9
3–4	1.9	5.6	15.9	6.0	7.8	6.6
4–5	3.8	10.0	15.1	11.2	4.2	16.9
5–6	9.7	17.1	17.8	12.9	13.5	19.2
6–7	23.5	17.7	20.1	18.1	31.2	20.3
7–8	13.8	16.5	7.4	24.1	17.1	16.6
8–9	3.5	10.3	4.3	9.5	9.1	8.9
9–10	8.6	8.3	2.4	5.2	5.5	4.2
10–11	14.9	5.0	2.1	4.3	4.4	3.8
11–12	16.8	1.8	0.4	0.9	2.1	1.1
12–13	2.4	0.3	0.2	0.9	0.5	0.4
13–14	0.3	0.3	0.0	0.0	0.0	0.0
14–15	0.0	0.0	0.0	0.0	0.0	0.0
15–16	0.0	0.3	0.0	0.0	0.0	0.0
Axle load (t)	TT 3 + 3	ST 2 + 1	ST 2 + 2	ST 2 + 3	ST 3 + 2	ST 3 + 3
<1	0.0	0.0	0.0	0.0	0.0	2.1
1–2	0.0	0.5	0.0	0.2	1.4	2.1
2–3	0.0	3.1	7.7	3.7	3.9	6.3
3–4	16.7	13.5	26.8	7.5	11.4	12.5
4–5	16.7	14.6	12.7	8.4	9.4	14.6
5–6	33.3	25.0	17.7	13.2	13.9	22.9
6–7	25.0	15.1	17.3	28.3	21.4	6.3
7–8	8.3	8.9	5.0	21.7	18.9	12.5
8–9	0.0	7.8	5.9	4.4	8.3	10.4
9–10	0.0	3.1	1.4	3.9	3.3	8.3
10–11	0.0	4.2	2.7	4.0	4.4	0.0
11–12	0.0	2.6	2.3	2.9	1.7	0.0
12–13	0.0	0.5	0.5	1.1	1.9	2.1
13–14	0.0	1.0	0.0	0.3	0.0	0.0
14–15	0.0	0.0	0.0	0.1	0.0	0.0
15–16	0.0	0.0	0.0	0.0	0.0	0.0

adjusted. The tire footprint width generally equals the tread width, except for strongly overinflated or ‘underloaded’ conditions, when the footprint width may be less than the tread width. The footprint length of a certain tire type (i.e. in case of constant size, manufacturer, diameter, structure) is rather constant with respect to combinations of load and pressure.

In Great Britain the tire pressure of 1,322 trucks, tractors and semi trailers were checked in 1998. The figures of four different checkpoints in the road network were averaged. The tire pressure values were identified according to the different axes: steering axle, driving axle (inner and outer side of twin tire assembly), second or

Table 5.19 Wheel loads distribution in percent on the “A 10 Tauern Autobahn”

Wheel load (t)	1998	1999	2000	2001	2002
<0.25	0.01	0.01	0.01	0.02	0.02
0.25–0.75	0.33	0.37	0.32	0.33	0.34
0.75–1.25	2.52	2.51	2.50	2.63	2.47
1.25–1.75	9.23	9.38	9.27	9.48	9.39
1.75–2.25	12.42	12.87	12.95	13.02	13.23
2.25–2.75	18.75	19.04	19.00	18.56	18.86
2.75–3.25	21.71	21.48	21.62	21.44	21.38
3.25–3.75	25.26	24.60	24.81	25.06	24.74
3.75–4.25	8.22	8.15	8.03	7.98	8.02
4.25–4.75	1.39	1.42	1.35	1.32	1.36
4.75–5.25	0.12	0.13	0.12	0.13	0.14
5.25–5.75	0.02	0.02	0.02	0.02	0.02
5.75–6.25	0.02	0.02	0.02	0.02	0.02
> 6.25	0.00	0.00	0.00	0.00	0.00
Wheel load (t)	2003	2004	2005	2006	Av.
< 0.25	0.02	0.01	0.01	0.01	0.01
0.25–0.75	0.33	0.28	0.42	0.31	0.34
0.75–1.25	2.68	2.00	2.60	2.37	2.49
1.25–1.75	9.43	9.08	9.49	9.36	9.34
1.75–2.25	13.04	12.83	13.29	13.06	12.96
2.25–2.75	18.46	18.78	18.80	18.79	18.78
2.75–3.25	21.27	21.74	21.25	21.50	21.49
3.25–3.75	25.13	25.66	24.54	24.96	24.98
3.75–4.25	8.11	8.18	8.10	8.12	8.10
4.25–4.75	1.37	1.28	1.33	1.34	1.35
4.75–5.25	0.14	0.12	0.13	0.13	0.13
5.25–5.75	0.02	0.02	0.02	0.02	0.02
5.75–6.25	0.01	0.02	0.02	0.02	0.02
>6.25	0.00	0.00	0.00	0.00	0.00

Table 5.20 Distribution of tire types according to COST 334 [3]

Tire type	Number of samples	Relative number (%)
425/65R 22.5	50	1
385/65R 22.5	1,404	34
315/80R 22.5	324	8
295/80R 22.5	337	8
275/70R 22.5	82	2
11R 22.5	60	1
12R 22.5	118	3
365/80R 20	523	12
Other 22.5	99	2
Other 19.5 and 17.5	602	15
Unknown	600	14

Table 5.21 Distribution of tire types for truck and tractor, trailer and semi trailer in percent of each according to COST 334 [3] including footprint width of tires

Tire type	Width (mm)	Footprint width (mm)	Truck and tractor (%)		Trailer and Semi trailer (%)
			Front axle	Rear axle	
385/65R22.5	385	285	5.6	0.8	–
315/80R22.5	315	247	36.5	31.5	–
295/80R22.5	295	235	36.0	40.7	–
275/70R22.5	275	224	8.6	7.7	–
11R22.5	279	184	3.0	4.5	–
12R22.5	305	201	10.3	14.8	–
425/65R22.5	425	308	–	–	2.4
385/65R22.5	385	285	–	–	74.3
365/80R20	365	274	–	–	23.3

towed (lift-) axle on truck or tractor, twin assembly on towed semitrailer axle, (super-) single tires on towed semitrailer axle.

Since the survey of the A 10 test section is focused on heavy trucks, the values of single trucks with only two axles were ignored. Hence, the following average pressure values determined of n values were considered:

Steering axle tires ($n = 1,560$):	8.47 bar
Driving axle, twin tires, outside ($n = 1,827$):	7.20 bar
Driving axle, twin tires, inside ($n = 945$):	7.14 bar
Towed truck or tractor single tires ($n = 374$):	7.66 bar
Semitrailer (super-) single tires, average of all three axles ($n = 3,004$):	8.57 bar
Semitrailer, twin tires, average inner and outer tire ($n = 1,131$):	7.00 bar

By using tire type and average tire pressure from COST 334 project [3] together with traffic census and axle load weight measurement from the field observation the wheel load distribution for this project can be estimated. The results are presented in Table 5.22. Footprint width and length depending on wheel load and tire pressure are listed in Table 5.23 for different axles, tire configurations and pressures.

5.4.5 Temperature

5.4.5.1 Air Temperatures

Air temperatures at the test section were measured every hour. The analysis of these data resulted in the average temperatures given in Table 5.24 and to extended data evaluation of summer temperatures as shown in Table 5.25 and Fig. 5.8.

On average, 18 days were tropical days (maximum temperature $T > 30^{\circ}\text{C}$) and 59 days summer days (maximum temperature $T > 25^{\circ}\text{C}$). On summer days the air temperature was on average 8.2 h above 25°C . From Table 5.25 follows that in an average year the total time with $T > 25^{\circ}\text{C}$ was 484 h. The average maximum temperature of the seven hottest days (week) according SHRP was $32.0 \pm 1.4^{\circ}\text{C}$.

Table 5.22 Wheel load distribution in percent on the “A 10 Tauern Autobahn”

Wheel load (t)	Steering axle 8.5 bar	Driving axle, twin tires 7.2 bar	Towed truck or tractor axle, single tires 7.7 bar	Semi trailer axle, single tires 8.5 bar
<0.25	—	—	—	—
0.25–0.75	—	0.1	0.2	—
0.75–1.25	0.1	0.8	0.9	0.7
1.25–1.75	0.1	3.5	1.8	4.0
1.75–2.25	0.3	5.1	2.5	5.1
2.25–2.75	1.3	8.6	3.6	5.3
2.75–3.25	6.1	4.2	3.2	8.0
3.25–3.75	10.9	0.3	2.8	11.0
3.75–4.25	3.4	—	1.7	2.9
4.25–4.75	0.4	—	0.5	0.4
4.75–5.25	—	—	0.1	—
>5.25	—	—	—	—
Total	22.6	22.6	17.3	37.5

Table 5.23 Footprint width and length depending on wheel load and tire pressure

(a) Steering axle, tire pressure = 8.5 bar						
Tire type R22.5	385/65	315/80	295/80	275/70	11	12
Width (mm)	285	247	235	224	184	201
Wheel load (kN)	Footprint length (mm)					
5	21	24	25	26	32	29
10	41	48	50	53	64	59
15	62	71	75	79	96	88
20	83	95	100	105	128	117
25	103	119	125	131	160	146
30	124	143	150	158	192	176
35	144	167	175	184	224	205
40	165	191	200	210	256	234
45	186	214	225	236	288	263
50	206	238	250	263	320	293
(b) Driving axle, twin tires, tire pressure = 7.2 bar						
Tire type R22.5	385/65	315/80	295/80	275/70	11	12
Width (mm)	285	247	235	224	184	201
Wheel load (kN)	Footprint length (mm)					
5	24	28	30	31	38	35
10	49	56	59	62	75	69
15	73	84	89	93	113	104
20	97	112	118	124	151	138
25	122	141	148	155	189	173
30	146	169	177	186	226	207
35	171	197	207	217	264	242
40	195	225	236	248	302	276
45	219	253	266	279	340	311
50	244	281	296	310	377	345

(continued)

Table 5.23 (continued)

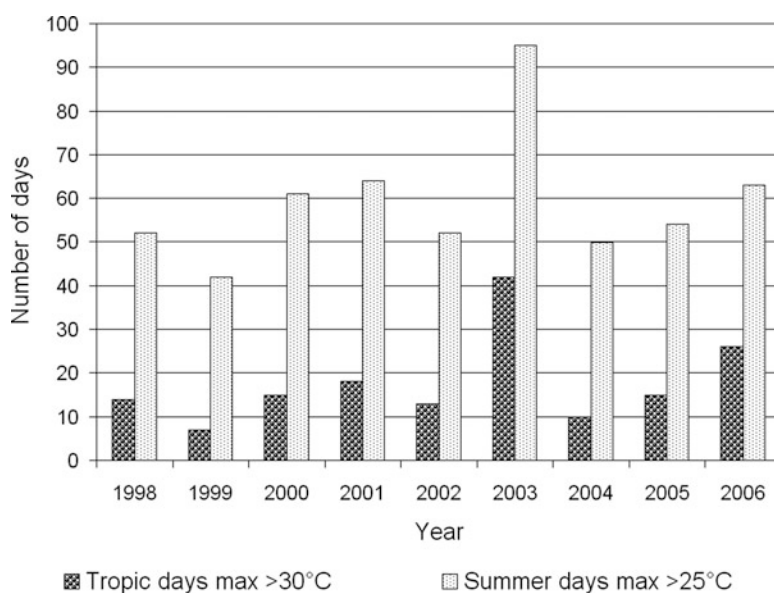
(c) Towed truck or tractor axle, single tires, tire pressure = 7.7 bar						
Tire type R22.5	385/65	315/80	295/80	275/70	11	12
Width (mm)	285	247	235	224	184	201
Wheel load (kN)	Footprint length (mm)					
5	23	26	28	29	35	32
10	46	53	55	58	71	65
15	68	79	83	87	106	97
20	91	105	111	116	141	129
25	114	131	138	145	176	162
30	137	158	166	174	212	194
35	159	184	193	203	247	226
40	182	210	221	232	282	258
45	205	237	249	261	318	291
50	228	263	276	290	353	323
(d) Semi trailer axle, single tires, tire pressure = 8.5 bar						
Tire type	425/65R22.5		385/65R22.5		365/80R20	
Width (mm)	308		285		274	
Wheel load (kN)	Footprint length (mm)					
5	19		21		21	
10	38		41		43	
15	57		62		64	
20	76		83		86	
25	95		103		107	
30	115		124		129	
35	134		144		150	
40	153		165		172	
45	172		186		193	
50	191		206		215	

Table 5.24 “A 10 Tauern Autobahn”, average air temperatures 1998–2007

Month	Air temperatures (°C)				
	Maximum	Av. maximum	Average	Av. minimum	Minimum
January	13.2	1.7	−2.8	−6.2	−19.7
February	21.2	6.1	0.2	−4.3	−15.4
March	23.2	11.4	5.1	−0.2	−17.8
April	26.9	14.9	9.5	4.5	−4.4
May	32.6	21.0	15.0	9.2	0.9
June	35.2	24.9	18.7	12.7	2.2
July	36.2	26.0	19.7	14.1	6.2
August	36.6	25.2	19.2	14.1	7.0
September	33.4	20.3	14.5	9.7	2.2
October	25.0	15.2	10.2	6.5	−3.6
November	20.4	7.6	3.8	0.8	−9.9
December	11.1	1.7	−1.6	−4.2	−15.0

Table 5.25 “A 10 Tauern Autobahn”, annual average number of hours with temperatures above a certain air temperature

Air temperature (°C)	Average number of hours	Air temperature (°C)	Average number of hours
37	0	30	39 ± 17
36	1 ± 2	29	49 ± 18
35	1 ± 3	28	59 ± 15
34	5 ± 7	27	76 ± 20
33	10 ± 8	26	87 ± 17
32	17 ± 8	25	114 ± 20
31	26 ± 13		

**Fig. 5.8** “A 10 Tauern Autobahn”, number hot days, depending on daily maximum temperature over the years

5.4.5.2 Pavement Temperature

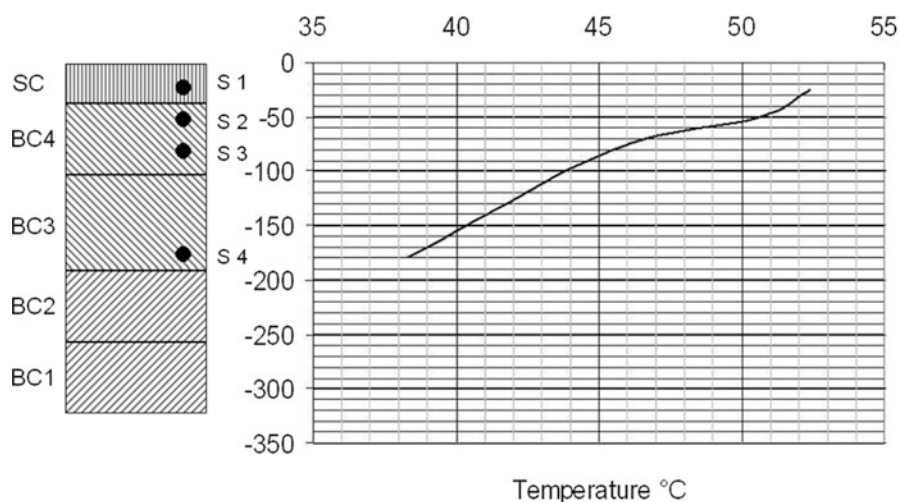
The temperatures in the pavement were measured every hour at four different locations below the surface

S1	25 mm below surface	(SC)	wearing course
S2	50 mm below surface	(BC4)	base course upper layer
S3	80 mm below surface	(BC4)	base course upper layer
S4	180 mm below surface	(BC2/3)	base course between 2nd and 3rd layer.

Absolute maximum and average of the yearly maximum temperatures measured at different pavement depths are given in Table 5.26. These data and the curve for

Table 5.26 “A 10 Tauern Autobahn”, maximum temperature and the average annual maximum temperature in the measuring points

	Max. temperature (°C)	Av. annual max. temperature (°C)
S 1	52.4	50.7
S 2	50.6	48.0
S 3	45.5	43.9
S 4	38.4	36.4

**Fig. 5.9** “A 10 Tauern Autobahn” Asphalt pavement, measuring points and maximum pavement temperatures

the maximum pavement temperatures as a function of pavement depth in Fig. 5.9 show that even at a depth of 180 mm an average maximum temperature of 36.4°C may be obtained and that the pavement temperature on the surface may well reach around 54°C in this test section.

Figure 5.10 compares the average annual number of hours in the measuring points for pavement temperatures >30°C at different depths. It follows, that the influence of pavement depth on the number of hours at certain temperatures decreases with decreasing temperatures.

From the temperature data it was also found that the maximum temperature in the wearing course of the test section (surface below 25 mm) according to SHRP was 53.07°C for a confidence level of 98%.

5.4.6 Rut Depth Measurements

Rut depth was measured annually in the ten cross profiles shown in Fig. 5.5. The final measurements in 2006/2007 are depicted in Fig. 5.11. It was found that the

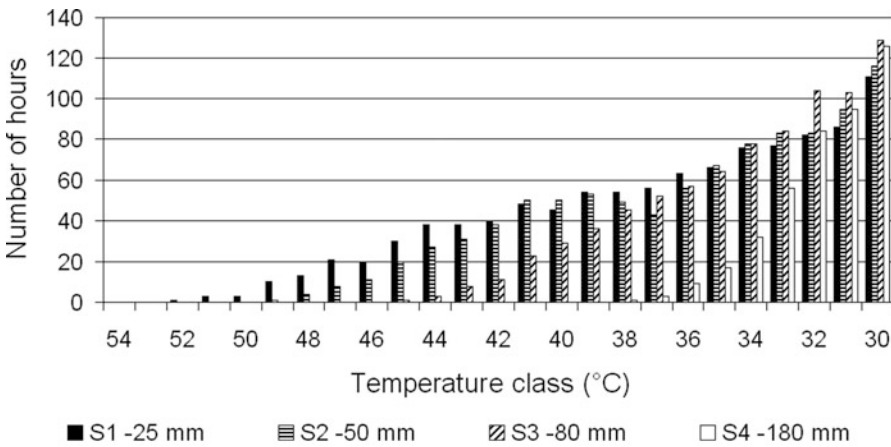


Fig. 5.10 “A 10 Tauern Autobahn”, pavement temperatures >30°C, average annual number of hours in the measuring points

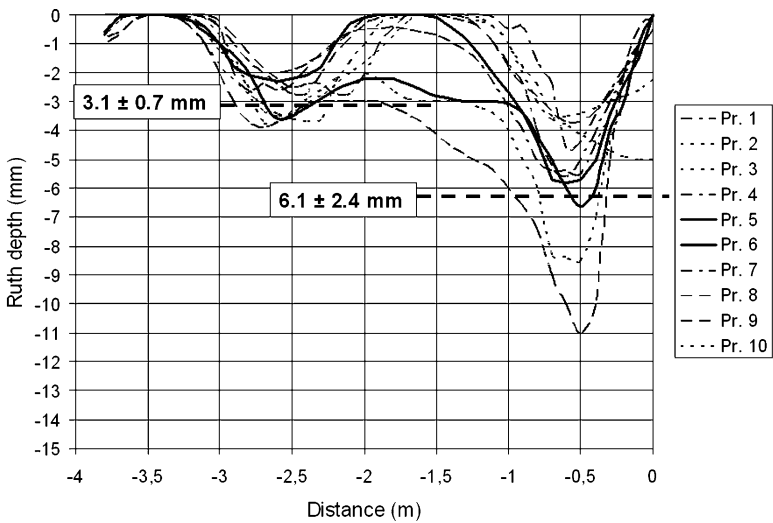


Fig. 5.11 “A 10 Tauern Autobahn”, final rut depth measurement 2006/2007

average rut depth in the right wheel track of the right lane was 6.1 mm (± 2.4 mm) and in the left wheel track 3.1 mm (± 2.4 mm). Cross profile 9 showed the deepest (11 mm) and profile 1 the least (4.6 mm) rutting. The profiles 5 and 6 at the end of the sampling zone were quite close in terms of maximum rut depths and their mean value of 6.2 mm was almost identical to the overall average.

Taking a pavement sample in the last year of observation revealed that 82% of rutting occurred in the SMA 11 wearing course. As shown in Table 5.27, it was also found that rutting evolution in the observed time period was almost linear, i.e. 0.22 mm per year.

Table 5.27 “A 10 Tauern Autobahn”, development of the average maximal rut depth in mm 1998–2006

Date of measurement	Average of profile 1 to 10		Average of profile 5 and 6	
	Left track	Right track	Left track	Right track
09.11.1998	−1.86	−4.42	−2.10	−4.50
09.12.1999	−2.15	−4.53	−2.40	−4.75
04.12.2000	−2.32	−4.71	−2.45	−5.00
06.12.2001	−2.41	−5.07	−2.55	−5.20
17.12.2002	−2.59	−5.32	−2.60	−5.40
17.12.2003	−2.72	−5.63	−2.70	−5.60
14.12.2004	−2.83	−5.62	−2.75	−5.80
18.10.2005	−2.95	−5.90	−2.85	−6.05
20.11.2006	−3.07	−6.09	−2.95	−6.20

5.5 Data Evaluation

5.5.1 Evaluation of Laboratory Test

5.5.1.1 Asphalt Properties

The results of asphalt testing presented in Sect. 5.3.2 can be used to calculate further asphalt properties. The complex modulus or stiffness modulus for various frequencies and temperatures was evaluated from the master curve. In cases where the air voids changes, the stiffness modulus was back calculated using the following equations proposed by Francken et al. [8] and Francken and Clauwaert [6].

Stiffness modulus:

$$E^* = E_{\infty} \cdot R^*$$

$$R^* = 10^{\log R}$$

$$\log R = \log \frac{B^*}{B_{\infty}} \cdot \left(1 - \left(1.35 \cdot \left(1 - e^{-0.13 \frac{V_{\text{MIN}}}{V_{\text{BIT}}}} \right) \right) \cdot \left(1 + \left(0.11 \cdot \log \frac{B^*}{B_{\infty}} \right) \right) \right)$$

$$E_{\infty} = 14,500 \cdot \left(\frac{V_{\text{MIN}}}{V_{\text{BIT}}} \right)^{0.55} \cdot e^{-0.0584 \cdot V_{\text{HR}}} \quad (5.7)$$

The corresponding notations are as follows

E^* = Stiffness modulus of asphalt (MPa)

E_{∞} = Max. stiffness modulus of asphalt (Glassy asphalt modulus) (MPa)

R^* = Reduced Modulus ($0 < R^* < 1$), describing the shape of the masters curve

B^* = Stiffness modulus of the binder (MPa)

B_{∞} = Max. stiffness modulus of the binder (Glassy binder modulus) (MPa)

V_{MIN} = Mineral content in the asphalt mixture (% by volume)

V_{BIT} = Binder content (% by volume)

V_{HR} = Air voids content (% by volume)

Relative vertical deformation and deformation rate (dynamic triaxial compression test) result from

$$\begin{aligned}
 E_p &= E^* \cdot f_v = E^* \cdot \frac{(2.716 + (1.432 \cdot V_{AV}))}{1,000} \\
 \varepsilon_v &= \frac{\sigma_v - \sigma_H}{2 \cdot E_p} \cdot \left(\frac{N}{450 \cdot v} \right)^{0.25} \\
 f_{c*} &= \frac{\varepsilon_{v2} - \varepsilon_{v1}}{N_2 - N_1} \cdot 10,000
 \end{aligned} \tag{5.8}$$

Here, the following notations are used:

E^* = Stiffness modulus of asphalt (MPa)

E_p = Deformation modulus of asphalt (MPa)

f_v = Factor deformation

V_{AV} = Air void content (% by volume)

σ_v = Vertical stress (MPa)

σ_H = Horizontal stress (MPa)

ε_v = Relative vertical deformation (%)

N = Number of load changes (–)

v = Traffic speed (km/h)

f_{c*} = Relative deformation rate ($\mu\text{m}/\text{m}/\text{n}$), where n is the increment of N

5.5.1.2 Comparison of the Dynamic Test

In this section, the type of test, test condition and test results are compared. In detail are covered: the wheel tracking test with the large device, the dynamic creep test, the dynamic indirect tensile test and the axial loading slab test. Note that samples with wearing course (SC) considered only!

The results from the different labs for different tests and conditions are presented in Tables 5.28, 5.29 and 5.30. For comparison of the different test results, the evolution of the rut depth or deformation with increasing number of loadings are expressed by power functions of the form

$$s = a \cdot N^b \tag{5.9}$$

where:

s = Rut depth or vertical deformation (mm)

N = Number of loadings

a, b = Parameters

In order to compare the WTT results obtained by the different laboratories from testing different layer configurations, a simple rough estimate of deformation was

Table 5.28 Wheel Tracking Test – large device, loading conditions, parameter of the deformation curve and deformation after 50,000 loadings (mm)

Laboratory	BE		CH		DK
Load (kN)	5				
Pressure (kPa)	600				
Loading time (s)	0.126				0.209
Test temperature (°C)	35	45	38	60	39 ^a
Sample thickness (mm)	32	32	100	100	260
Parameter					
a	0.310	0.355	0.099	0.115	0.081
b	0.188	0.172	0.266	0.456	0.276
R ²	0.981	0.999	0.967	0.967	0.871
Deformation after 50,000 loadings (mm)	2.37	2.29	1.75	16.09	1.60

^aTest temperature 40°C on the surface and 20°C on the bottom of the sample**Table 5.29** Dynamic Creep Test and Axial Loading slab Test, loading conditions, parameter of the deformation curve and deformation after 50,000 loadings (mm)

Laboratory	DK	ZA		
Load (kN)	0.79	5.5		
Pressure (kPa)	100	700		
Loading time (s)	1	0.2		
Test temperature (°C)	40	30	40	50
Sample thickness (mm)	200	325 ^a	325 ^a	325 ^a
Parameter				
a	0.363	0.007	0.034	0.047
b	0.135	0.448	0.352	0.389
R ²	1	0.999	0.994	0.993
Deformation after 50,000 loadings (mm)	1.57	0.84	1.92	3.16

^aSamples are placed on synthetic support**Table 5.30** Dynamic Indirect Tensile Test, loading conditions, parameter of the deformation curve and deformation after 50,000 loadings (mm)

Laboratory	DE 2			
Load (kN)	6.3			
Pressure (kPa)	700			
Loading time (s)	0.02			
Test temperature (°C)	25	34	40	44
Sample thickness (mm)	200	200	200	200
Parameter				
a	0.018	0.124	0.084	0.040
b	0.218	0.213	0.300	0.373
R ²	0.897	0.950	0.972	0.975
Deformation after 50,000 loadings (mm)	0.19	1.25	2.18	2.24

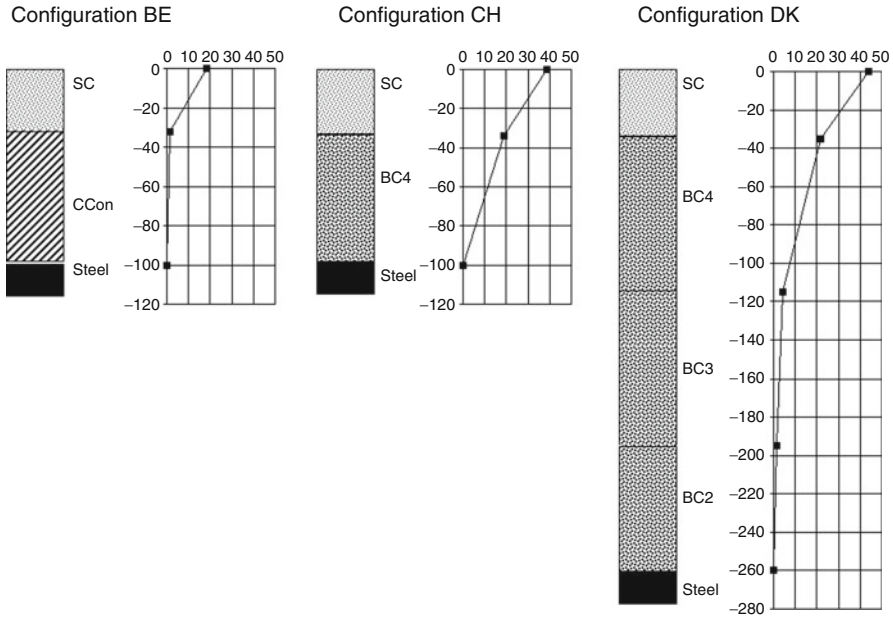


Fig. 5.12 Deformation in μm during Wheel Tracking Test at about 40°C surface temperature at various test configurations, estimated with computer code BISAR [1]

performed with the computer code BISAR [1] and the master curves given in Table 5.5 for one loading cycle, being well aware that the boundary conditions are not strictly comparable and matching the assumptions of the program. The results in Fig. 5.12 demonstrate that the influence of the loading conditions on the rutting test results may be very strong. This is particularly true when testing thin courses only. Hence, for assessing rutting of real pavement slabs it is proposed to test slabs at least containing 100 mm of the top part of the pavement.

In order to provide an answer to the critical temperature when significant rutting has to be expected, the deformations after 50,000 loadings as given in Tables 5.28 and 5.30 may be considered. As shown in Fig. 5.13, the Cyclic Indirect Tensile Test (ITT-DE2) and the Axial Load slab Test (ALT-ZA) both carried out with an axial pressure of 700 kPa are in very good agreement. ($R^2 = 0.97$). The WTT showed considerable scatter. The graphic show that from all the data, a rut depth of 1 mm may be expected after 50,000 loadings at a temperature of 32°C . This temperature is in good agreement with the average maximum temperature of the seven hottest days according SHRP. Therefore the temperature limit in the wearing course of 32°C was defined for the temperature and traffic evaluation of the test section.

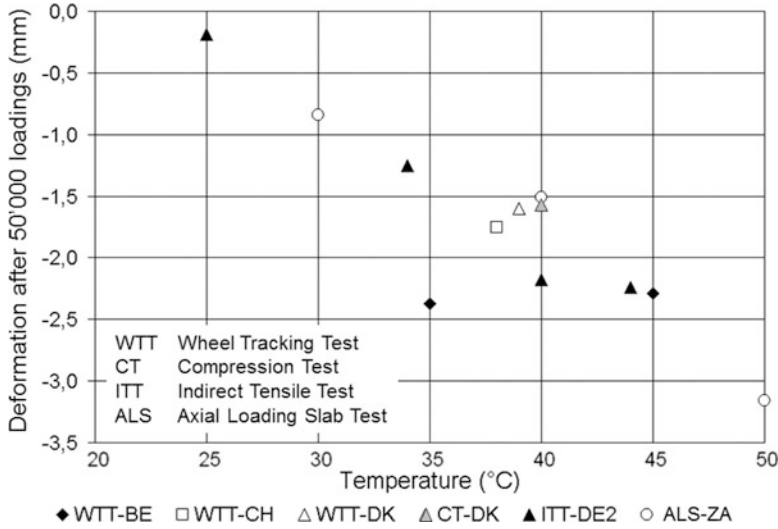


Fig. 5.13 Rut depth and temperatures after 50,000 loadings calculated for various tests using a power law approach

5.5.2 Evaluation of Traffic Data

5.5.2.1 Wheel Track Distribution

The wheel track changes on the surface of a road depending on the width of the lane, the width of the truck and the width of the tire. Experience and investigations showed that the distribution follows the Gaussian normal distribution. Therefore standard deviation STD can be calculated with:

$$STD = \frac{\text{width of the lane} - \text{width of the truck}}{6} \quad (5.10)$$

The width of the lane is 3.75 m and the width of a truck is about 2.5 m. Therefore, according to the above equation $STD = 208$ mm. The wheel track factor f_{wt} which weights the occurrence of loading in a wheel track is calculated by means of the one-sided standardized Gaussian normal distribution as shown in Table 5.31 for the most common single twin and twin tire types. For example, in case of a single tire with a footprint width w_F the equation for f_{WT} reads

$$f_{wt} = 2 \times \int_{-\infty}^{\frac{w_F}{2 \times STD}} \frac{1}{2\pi} e^{-\frac{z^2}{2}} dz. \quad (5.11)$$

Table 5.31 Wheel track factor f_{WT} for single and twin tires

Tire type	Width (mm)		Single tire	Twin tire
	Tire	Footprint width (w_F)	f_{WT}	f_{WT}
385/65R22.5	385	285	0.50	0.57
315/80R22.5	315	247	0.45	0.57
295/80R22.5	295	235	0.42	0.57
275/70R22.5	275	224	0.40	0.57
11R22.5	279	184	0.34	0.47
12R22.5	305	201	0.38	0.48
425/65R22.5	425	308	0.54	–
385/65R22.5	385	285	0.50	–
365/80R20	365	274	0.49	–

5.5.2.2 Wheel Load Conditions

The loading time of a wheel load in a contact point of the pavement surface depends on the traffic speed and the length of the footprint calculated from the wheel load and the tire pressure. The speed of trucks on the test section was about 95 km/h (26.39 m/s).

Assuming an equivalent single axle load ESAL of 100 kN, the loading time varies from 0.0078 s (steering axle, wheel load 50 kN, tire pressure 8.5 bar, tire type 385/65R22.5) to 0.01430 s (driving axle, twin tires, wheel load 50 kN, tire pressure 7.2 bar, tire type 11R22.5).

The loading time corresponds to the frequency which is used for calculating the stiffness modulus from the master curve according Eq. (5.12).

$$f = \frac{1}{2 \times \pi \times t} \quad (5.12)$$

where:

f = frequency (Hz)

t = loading time (s)

5.5.2.3 Influence of Wheel Load Conditions on Rutting

The influence of wheel load conditions on rutting was investigated theoretically by using the tests results and the master curve of the wearing course (SC). The reference temperature chosen for this study was 45°C and the frequency f calculated from the loading time t was taken from Sect. 5.5.2.2. In this way the stiffness modulus E^* could be calculated.

The relative deformation $\varepsilon_v (10,000)$ and relative deformation rate $f^*_{c (10,000-5,000)}$ was calculated based on a theoretic dynamic triaxial compression test. The deformation modulus E_P was derived from the stiffness modulus E^* applying Eqs. (5.7)

Table 5.32 Relative deformation and deformation rate, wearing course (SC), temperature = 45 °C, vertical stress = 0.85 MPa, V = 95 km/h

Tire type	425/ 65R22.5	385/ 65R22.5	365/ 80R20	425/ 65R22.5	385/ 65R22.5	365/ 80R20
Wheel load (kN)	Relative deformation $\epsilon_v(10,000)$ (%)			Relative deformation rate $f^*_{c(10,000-5,000)}$ (µm/m/n)		
5	2.79	2.88	2.91	0.89	0.92	0.93
10	3.59	3.69	3.75	1.14	1.18	1.19
15	4.15	4.26	4.32	1.32	1.35	1.38
20	4.57	4.69	4.75	1.45	1.49	1.51
25	4.92	5.03	5.11	1.56	1.60	1.63
30	5.20	5.33	5.38	1.65	1.70	1.71
35	5.48	5.60	5.65	1.75	1.78	1.80
40	5.71	5.84	5.91	1.82	1.86	1.88
45	5.91	5.98	6.05	1.88	1.90	1.93
50	6.05	6.21	6.21	1.93	1.98	1.98
55	6.21	6.38	6.38	1.98	2.03	2.03

and (5.8) for an air void content $V_{AV} = 2.3\%$ by volume. The vertical stress was assumed from the tire pressure, the horizontal stress was estimated to be 0.1 MPa and the traffic speed was 95 km/h.

As an example, Table 5.32 shows the results for the wheels of the semitrailer axle.

This calculation was carried out for all types of tires and tire pressures. One major conclusion is that the type of tire is not influencing rutting behavior significantly, whereas wheel load and tire pressure have to be taken into account. A standard wheel configuration can simplify the complex calculations. For this purpose the semi trailer truck 2 + 3 representing 53% of heavy traffic was chosen, taking into account that 75% were semi trailer wheels of type 385/65R22.5. Hence, the standard wheel configuration WC_S related to the standard wheel load SWL which was used in this study as reference for the equivalent standard wheel load $ESWL$ reads

Standard tire type TT_S :	385/65R22.5
Standard tire pressure p_S :	8.5 bar
Standard traffic speed v_S :	95 km/h
Standard wheel load SWL :	50kN

The corresponding deformation characteristics of the wearing course (SC) at reference temperature 45°C are:

Relative deformation $\epsilon_v(10,000)$, $SWC,ESWL$	6.21%
Relative deformation rate $f^*_{c(10,000-5,000)}$, $SWC,ESWL$	1.98 (µm/m/n)
Relative deformation rate factor DF	1.00

The relative deformation rate factor is defined as the ratio between the relative deformation rate $f^*_{c(10,000-5,000)}(TT_X, p_X, v_X, WL_X)$ due to any chosen wheel “X”

Table 5.33 “A 10 Tauern Autobahn” ESWL’s and truck axles

Wheel load (kN)	Steering axle	Driving axle	Towed axle	Semi trailer axle
5	0.22	0.26	0.21	0.23
10	0.31	0.36	0.29	0.30
15	0.35	0.40	0.32	0.34
20	0.38	0.44	0.35	0.38
25	0.41	0.47	0.37	0.41
30	0.43	0.49	0.39	0.43
35	0.45	0.51	0.41	0.45
40	0.47	0.53	0.43	0.47
45	0.48	0.55	0.44	0.48
50	0.50	0.56	0.45	0.50
55	0.51	0.57	0.46	0.51

with a certain tire pressure p_X , traffic speed v_X and wheel load WL_X divided by the relative deformation rate $f_{c(10,000-5,000)}^*(TT_S, p_S, v_S, SWL)$ due to the standard wheel configuration. By definition, the relative deformation rate factor for the standard wheel configuration WC_S under SWL is 1.00.

$$DF_X = \frac{f_{c(10,000-5,000)}^*(TT_X, p_X, v_X, WL_X)}{f_{c(10,000-5,000)}^*(TT_S, p_S, v_S, SWL)} \quad (5.13)$$

5.5.2.4 Evaluation of the Equivalent Standard Wheel Load (ESWL)

With the relative deformation rate factor from Eq. (5.13) the number of equivalent standard wheel loads $ESWL$ ’s for any chosen wheel “X” with the wheel track factor $f_{WT''X''}$ and the corresponding percentage of occurrence of the wheel $n_{wheel''X''}$, equivalent to the standard wheel load SWL and the standard wheel configuration WC_S were calculated as follows.

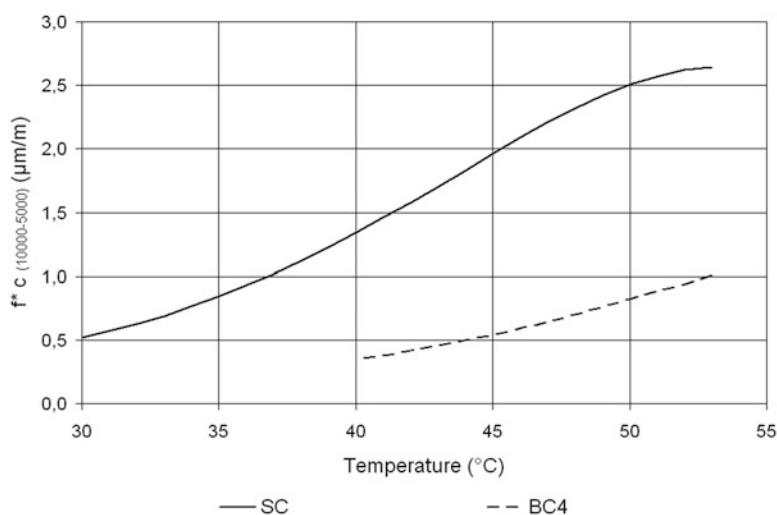
$$ESWL's = DF_X \times f_{WT''X''} \times n_{wheel''X''} \quad (5.14)$$

The ESWL’s for different wheel loads and truck axles are given in Table 5.33. With help of the wheel load distribution and the axle configuration, the ESWL’s for each type of truck was evaluated (Table 5.34).

Considering the average proportion of heavy traffic, one single truck corresponds to 4.04 ESWL in this study. The average wheel track factor is 0.45. This results on average in 1.82 ESWL for the 3.75 m wide lane.

Table 5.34 “A 10 Tauern Autobahn”, truck types, ESWL’s, wheel track factor and ESWL’s in the lane

Truck type	ESWL’s	f_{WT}	ESWL’s/lane
Truck T2	1.87	0.48	0.90
Truck T3	2.69	0.46	1.24
Truck T4	3.39	0.45	1.53
Trailer truck TT2 + 2	3.64	0.45	1.64
Trailer truck TT2 + 3	4.64	0.44	2.06
Trailer truck TT3 + 2	4.57	0.44	2.03
Trailer truck TT3 + 3	4.69	0.44	2.07
Semitrailer truck STT2 + 1	2.57	0.49	1.26
Semitrailer truck STT2 + 2	3.43	0.45	1.55
Semitrailer truck STT2 + 3	4.63	0.44	2.05
Semitrailer truck STT3 + 2	4.56	0.44	2.02
Semitrailer truck STT3 + 3	5.44	0.44	2.38

**Fig. 5.14** Relative deformation rate of the wearing and base course loaded by the equivalent standard wheel load

5.5.2.5 Influence of the Asphalt Temperature

The effect of the ESWL’s on rutting depends on the temperature of the asphalt pavement. The relative deformation rate of the standard wheel configuration was calculated theoretically for temperatures between 30 and 53°C as shown in Fig. 5.14.

Taking into account that the reference temperature T_R chosen in this study was 45°C, the number of equivalent standard wheel load $ESWL_{45}$ ’s for each temperature in the wearing course can be calculated with the factor $ESWL’s(T)$ and the relative deformation rate factor $DF_{TR}(T)$, i.e. the ratio of the relative deformation rates of

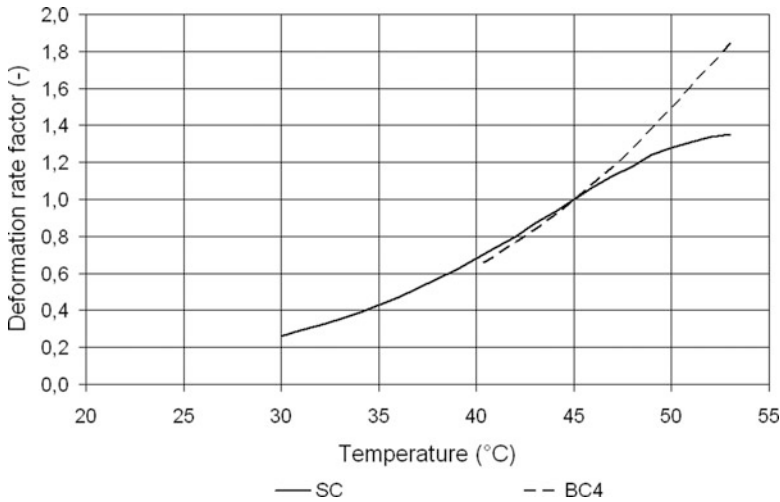


Fig. 5.15 Relative deformation rate factor of the wearing course and base course normalized to the relative deformation rate at 45°C (=1.00)

standard configuration $f_{c(10,000-5,000)}^*(T)$ at an arbitrary temperature and $f_{c(10,000-5,000)}^*(T_R)$ at the reference temperature.

$$\begin{aligned} \text{ESWL}_{45}'s &= \text{ESWL}'s(T) \times \frac{f_{c(10,000-5,000)}^*(T)}{f_{c(10,000-5,000)}^*(T_R)} \\ &= \text{ESWL}'s(T) \times DF_{TR}(T) \end{aligned} \quad (5.15)$$

The result is shown in Fig. 5.15 for a reference temperature $T_R = 45^\circ\text{C}$. It can be seen that the relative deformation rate factor $DF_{45}(T)$ for both wearing and base course are quite similar below 45°C in the evaluated temperature range. It also shows that the increase of the relative deformation rate in the base course appears to be faster at higher temperatures as compared to the wearing course.

5.5.3 Influence of Traffic and Temperature on Rutting

5.5.3.1 Stresses and Deformations in the Asphalt Pavement on Hot Summer Days

The change of stresses and vertical deformations during a hot summer day in the upper part of the asphalt pavement was investigated with the BISAR – PC program [1]. As an example, August, 13 in 2003 was chosen. The measured temperatures are shown in Fig. 5.16. The maximum air temperature measured at 3 pm was 36.6°C and the temperatures in the upper part of the pavement almost hit the 50°C mark.

The material properties for the stress and strain calculation were derived from the laboratory and field tests. The elastic modulus of the asphalt was calculated

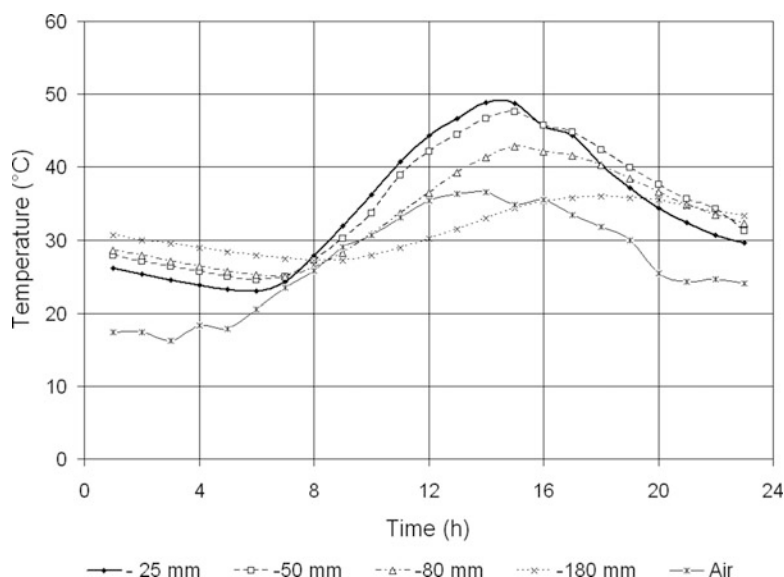


Fig. 5.16 Air temperatures and temperatures in the asphalt pavement on a hot summer day: 13th August 2003

from the master curve as given in Table 5.5 for a loading time of 20 Hz corresponding to a traffic speed of 95 km/h (Table 5.35). The load was 50 kN and the axial pressure 8.5 bar.

The results in Table 5.35 clearly demonstrate that the elastic modulus of the wearing course (SC) during hottest daytime was about half of the elastic modulus in the upper part in the 4th layer of the base course (BC4) and almost an order of magnitude below the modulus of the lowest asphalt layer BC1. From Table 5.36 follows that the vertical stresses in the wearing course were about 10% higher than in the upper base course BC4. The vertical deformations of the wearing course were in the same order of magnitude than determined with the WTT's (Fig. 5.12), However, the deformations in the base course were estimated about 25% higher in the WTT than in the pavement partly due to the uniform temperature distribution, suggesting that the WTT methods are not reflecting the real situation in a realistic way. The simple estimate also showed that the vertical stresses in the WTT as well as the ALS-ZA were about 40% lower and in case of the DART and ALS-ZA tests, where several layers of the pavement could be tested also faster decreasing than in the real pavement.

5.5.3.2 Temperatures in the Wearing Course and Numbers of Trucks

The laboratory tests depicted in Fig. 5.13 showed that the permanent deformation in the cyclic indirect tensile test (ITT-DE2) and the axial load slab test (ALT-ZA) after

Table 5.35 Temperatures and E modules for calculation

Time	10 am	12 am	2 pm	4 pm	6 pm	8 pm	10 pm
Temperatures °C							
Air	29.0	33.1	36.3	34.8	33.4	29.9	24.2
–20 mm	36.2	44.3	48.9	45.6	40.3	34.4	30.6
–50 mm	33.6	42.1	46.7	45.7	42.4	37.6	34.2
–80 mm	30.8	36.4	41.3	42.1	40.3	36.5	33.4
–180 mm	27.9	30.2	30.0	35.3	36.0	35.5	33.9
Elastic modulus (MPa)							
E ₁ , SC	1,541	774	580	677	1,067	1,868	2,503
E ₂ , BC4 up	3,535	1,738	1,131	1,229	1,738	2,484	3,535
E ₃ , BC4 low	4,563	2,967	1,900	1,738	2,077	2,715	3,854
E ₄ , BC3	7,294	6,409	5,197	4,471	4,132	4,132	4,826
E ₅ , BC2	5,269	4,797	3,934	3,180	3,180	3,180	3,543
E ₆ , BC1	6,252	5,806	5,377	4,966	4,966	4,966	4,966
E ₇ , road base	300						
E ₈ , sub base	240						
E ₉ , sub grade	140						

Table 5.36 Stresses and vertical deformations in the upper part of the asphalt pavement

		10 am	12 am	2 pm	4 pm	6 pm	8 pm	10 pm
Course	Depth (mm)	Vertical stress σ_v (MPa)						
	0	–0.85	–0.85	–0.85	–0.85	–0.85	–0.85	–0.85
SC	–35	–0.84	–0.84	–0.84	–0.84	–0.84	–0.84	–0.84
BC4 up.	–75	–0.77	–0.77	–0.78	–0.78	–0.78	–0.77	–0.76
BC4 low	–115	–0.62	–0.63	–0.64	–0.65	–0.64	–0.63	–0.61
BC3	–119	–0.30	–0.31	–0.32	–0.33	–0.33	–0.32	–0.30
Course	Depth (mm)	Vertical deformation (μm)						
	0	205	231	254	259	247	231	216
SC	–35	197	216	236	242	235	225	210
BC4 up.	–75	192	208	222	230	226	218	206
BC4 low	–115	188	202	214	221	218	212	201
BC3	–119	184	198	209	214	212	204	195

50,000 loadings at 32°C was about 1 mm. The axial pressure in both cases was 0.7 MPa whereas the loading time in the ITT-DE2 was 0.02 s and in the ALT-ZA 0.2 s. For these test conditions the values $\varepsilon_{50,000}$ and $f_{c(10,000-5,000)}^*$ were calculated. The same calculation carried out with an axial pressure of 0.85 MPa led to the same values at 30°C. Therefore, it was assumed that the numbers of trucks at temperatures higher than 30°C are relevant for rutting in the wearing course. These numbers of trucks are given in Table 5.37.

During the test period 727,231 trucks were counted at temperatures in the wearing course higher than 30°C.

Table 5.37 Annual numbers of trucks and temperatures in the wearing course

T (°C)	Year								
	1998	1999	2000	2001	2002	2003	2004	2005	2006
54	0	0	0	0	0	0	0	0	0
53	173	0	0	0	76	86	0	0	0
52	442	33	0	0	153	171	0	75	728
51	870	163	0	0	305	342	0	136	705
50	965	114	71	0	1,069	1,385	61	1,048	1,607
49	1,580	378	354	391	947	1,566	294	1,577	1,811
48	1,566	583	1,012	1,307	1,294	3,170	710	1,860	2,359
47	1,689	785	1,344	1,270	1,304	2,691	1,123	1,826	2,119
46	1,585	518	2,445	2,117	1,931	3,814	1,739	2,059	2,336
45	2,386	1,238	3,010	2,584	3,408	4,954	1,798	2,443	3,759
44	3,355	1,027	3,188	3,641	2,140	4,296	1,624	3,498	2,752
43	2,482	903	3,476	2,863	3,274	4,900	1,574	3,004	2,077
42	2,336	1,612	4,328	4,020	3,960	5,074	2,478	3,415	3,504
41	2,640	1,510	3,389	4,569	2,757	4,402	2,884	3,471	3,447
40	3,044	1,639	3,965	3,779	4,069	5,612	2,446	5,104	4,522
39	4,918	1,945	3,397	4,388	4,071	4,766	1,865	4,318	3,424
38	4,321	2,868	3,956	3,566	3,815	4,954	3,252	4,377	4,780
37	5,457	2,846	4,981	4,891	4,949	5,573	2,467	3,820	4,787
36	5,116	3,465	4,813	4,254	4,551	7,731	3,580	5,471	4,481
35	6,752	2,862	5,953	5,670	6,902	6,890	3,775	4,691	6,298
34	6,055	2,403	5,114	5,638	6,014	7,530	3,604	4,932	6,501
33	6,764	3,370	6,766	4,991	6,597	7,721	3,925	4,441	8,232
32	6,936	3,140	6,030	6,791	6,118	7,162	3,899	5,555	7,091
31	7,493	4,437	7,818	8,960	9,120	9,409	5,420	9,126	8,678
30	7,977	4,399	7,136	6,290	9,226	9,728	4,537	7,697	8,596
Total	86,902	42,235	82,542	81,979	88,051	113,927	53,056	83,946	94,595

Table 5.38 Average annual equivalent wheel load factor to 50 kN

Year	1998	1999	2000	2001	2002	2003	2004	2005	2006
$f_{EWL\ 50}$	1.79	1.78	1.80	1.83	1.82	1.85	1.84	1.81	1.81

Taking into account the truck types, their proportion, their ESWL's (Table 5.35), the average annual wheel loads factor f_{EWL50} equivalent to 50 kN for a truck was calculated as shown in Table 5.38.

In total 727,231 trucks correspond to 1,321,492 wheel loads equivalent to 50 kN.

In Table 5.39, the annual numbers of standard wheel loads equivalent to 50 kN at 45°C $ESWL_{45}$'s in the wearing course are presented. $DF_{45}(T)$ is the relative deformation rate factor of the wearing course normalized to the relative deformation rate at 45°C (Fig. 5.15). In total 751,364 loads of 50 kN and 0.85 MPa axial pressure were driving over the wearing course at 45°C.

Table 5.39 Annual numbers of wheel loads $ESWL_{45}$'s equivalent to 50 kN at 45°C in the wearing course

T (°C)	Year									
	DF_{45}	1998	1999	2000	2001	2002	2003	2004	2005	2006
54		0	0	0	0	0	0	0	0	0
53	1.35	418	0	0	0	187	213	0	0	0
52	1.34	1,057	78	0	0	371	422	0	182	1,765
51	1.31	2,043	381	0	0	729	828	0	323	1,677
50	1.28	2,211	260	162	0	2,485	3,267	142	2,430	3,726
49	1.24	3,498	833	784	884	2,129	3,572	667	3,534	4,058
48	1.18	3,321	1,230	2,151	2,834	2,786	6,926	1,542	3,993	5,063
47	1.13	3,409	1,577	2,720	2,621	2,674	5,596	2,322	3,731	4,330
46	1.07	3,024	983	4,678	4,132	3,742	7,500	3,398	3,979	4,513
45	1.00	4,276	2,209	5,408	4,736	6,202	9,147	3,300	4,431	6,818
44	0.93	5,621	1,712	5,355	6,239	3,641	7,416	2,787	5,933	4,667
43	0.87	3,865	1,399	5,428	4,561	5,178	7,864	2,510	4,736	3,274
42	0.80	3,368	2,314	6,256	5,929	5,799	7,538	3,658	4,985	5,114
41	0.74	3,510	1,998	4,518	6,213	3,722	6,029	3,927	4,671	4,639
40	0.68	3,721	1,994	4,860	4,725	5,051	7,069	3,063	6,317	5,597
39	0.62	5,506	2,167	3,813	5,024	4,628	5,497	2,138	4,893	3,880
38	0.57	4,418	2,919	4,055	3,729	3,961	5,219	3,405	4,530	4,947
37	0.52	5,089	2,641	4,656	4,665	4,686	5,355	2,356	3,606	4,518
36	0.47	4,341	2,926	4,094	3,691	3,921	6,758	3,110	4,698	3,849
35	0.43	5,204	2,195	4,599	4,470	5,402	5,471	2,979	3,660	4,914
34	0.39	4,237	1,673	3,587	4,035	4,274	5,429	2,583	3,493	4,605
33	0.35	4,293	2,129	4,305	3,240	4,252	5,048	2,551	2,853	5,288
32	0.32	3,992	1,799	3,479	3,998	3,576	4,247	2,298	3,237	4,131
31	0.29	3,911	2,305	4,091	4,784	4,834	5,060	2,897	4,822	4,585
30	0.26	3,776	2,073	3,387	3,046	4,435	4,745	2,199	3,689	4,119
Total		88,109	39,795	82,388	83,556	88,665	126,215	53,830	88,726	100,079

5.5.3.3 Temperatures in the Base Course (BC4) and Numbers of Trucks

The relevant temperature for rutting was calculated in the same way as in Sect. 5.5.3.2. The values $\varepsilon_{50,000}$ and $f_{c(10,000-5,000)}^*$ at 30°C of the wearing course were the basis. The axial pressure in the base course was reduced to 0.83 MPa. The result showed that the number of trucks at temperatures higher than 43°C were relevant for rutting in the base course (Table 5.40).

In this study the base course (BC4) was divided in an upper and a lower layer of 40 mm thickness. The temperatures of the measuring point 50 mm below the surface were considered relevant for the upper layer and the temperatures 80 mm below surface for the lower layer.

During the test period 91,333 trucks relevant for the upper layer of BC4 and 13,877 trucks relevant for the lower layer of B4 were counted. In terms of wheel loads equivalent to 50 kN this means 166,449 and 25,242 wheel loads respectively.

In Table 5.41 the annual numbers of standard wheel loads equivalent to 50 kN at 45°C $ESWL_{45}$'s in the base course are presented together with the relative deformation rate factor of the base course $DF_{45}(T)$.

Table 5.40 Annual numbers of trucks and temperatures in the base course (BC4)

T (°C)	Year								
	1998	1999	2000	2001	2002	2003	2004	2005	2006
Upper layer									
52	0	0	0	0	0	0	0	0	0
51	0	0	0	0	0	86	0	0	0
50	0	0	0	0	229	171	0	0	75
49	173	33	0	0	763	864	0	458	149
48	528	98	0	0	916	1,910	61	1,506	306
47	1,130	195	0	461	968	2,249	534	1,077	323
46	1,234	429	354	1,270	1,762	3,873	1,130	2,199	1,225
45	1,493	581	1,627	2,131	1,900	4,460	1,410	2,248	1,652
44	2,376	789	1,487	2,249	1,997	4,981	1,961	2,475	2,172
43	1,935	796	2,826	2,861	3,116	4,754	2,622	2,922	2,776
Total	8,869	2,920	6,293	8,972	11,652	23,347	7,717	12,885	8,678
Lower layer									
47	0	0	0	0	0	0	0	0	0
46	0	0	0	0	229	86	0	0	0
45	264	130	0	0	687	344	0	79	75
44	961	195	0	0	916	1,557	0	931	232
43	1,047	477	566	316	815	2,082	61	1,280	546
Total	2,272	803	566	316	2,647	4,069	61	2,290	853

Table 5.41 Annual number of wheel loads $ESWL_{45}$'s equivalent to 50 kN at 45°C in the base course (BC4)

T (°C)	DF ₄₅	Year								
		1998	1999	2000	2001	2002	2003	2004	2005	2006
Upper layer										
52	1.73	0	0	0	0	0	0	0	0	0
51	1.62	0	0	0	0	0	255	0	0	0
50	1.50	0	0	0	0	625	474	0	0	203
49	1.39	431	81	0	0	1,930	2,216	0	1,154	376
48	1.28	1,215	224	0	0	2,141	4,528	143	3,508	713
47	1.18	2,395	412	0	999	2,084	4,913	1,160	2,311	693
46	1.09	2,408	832	692	2,535	3,492	7,786	2,258	4,344	2,420
45	1.00	2,676	1,037	2,923	3,907	3,458	8,234	2,587	4,079	2,997
44	0.92	3,908	1,291	2,452	3,783	3,335	8,440	3,302	4,120	3,616
43	0.84	2,916	1,193	4,270	4,410	4,770	7,382	4,047	4,457	4,235
Total		15,949	5,070	10,337	15,634	21,833	44,228	13,496	23,973	15,253
Lower Layer										
47	1.18	0	0	0	0	0	0	0	0	0
46	1.09	0	0	0	0	454	172	0	0	0
45	1.00	473	232	0	0	1,250	635	0	143	135
44	0.92	1,580	320	0	0	1,530	2,639	0	1,549	386
43	0.84	1,579	716	855	488	1,248	3,233	93	1,953	834
Total		3,632	1,268	855	488	4,482	6,679	93	3,645	1,355

For the upper layer 165,746 and for the lower layer 22,497 loads of 50 kN and 0.85 kPa axial pressure were relevant in total for the base course BC4 at 45°C.

5.5.3.4 Temperatures in the Courses and Number of Heavy Vehicles

The number of standard wheel loads $ESWL_{45}$'s equivalent to 50 kN at 45°C influenced by the temperatures in the courses and the number of heavy vehicles can be estimated with Eq. (5.16). The rutting temperature which determines the factor of rutting temperature f_{TC} has to be determined by investigation or calculated. It is the temperature where after 50,000 loadings with pressure of 0.85 MPa a rut depth of 1 mm occurs.

$$ESWL_{45}'s = AADHT \times f_d \times d_{T_{max} > 25^\circ C} \times f_{TC} \times t \quad (5.16)$$

where:

$ESWL_{45}$'s = number of equivalent standard wheel loads of 50 kN at 45°C

AADHT = number of average annual daily heavy traffic

f_d = factor of direction (normally 0.5)

$d_{T_{max} > 25^\circ C}$ = number of average annual summer days warmer than 25°C

f_{TC} = rutting temperature factor of the course (from Fig. 5.17)

t = time, number of years

5.5.3.5 Rut Depth and Thickness of Pavement Affected by Rutting

The investigation in Sect. 5.4.6 showed that 82% of rutting occurred in the wearing course.

In total 751,364 loads of 50 kN and 0.85 MPa axial pressure at 45°C lead to rutting of the pavement in the wearing course. 165,746 loads or 22% of these total loads create ruts in the whole base course (BC4), mainly in the upper part, and 22,497 of these total loads or 3% cause also rutting in the lower part of the base course (BC4).

Therefore, it can be concluded that the wearing course and the upper part of the base course namely BC4 are damaged by rutting. The thickness of this part is 115 mm.

5.5.3.6 Rut Depths and Number of Wheel Loads Equivalent to 50 kN at 45 °C

The development of the average maximum rut depth in the right wheel track (Table 5.28) and the development of the wheel loads equivalent to 50 kN and 45°C (Tables 5.39, 5.40 and 5.41) are summarized in Table 5.42.

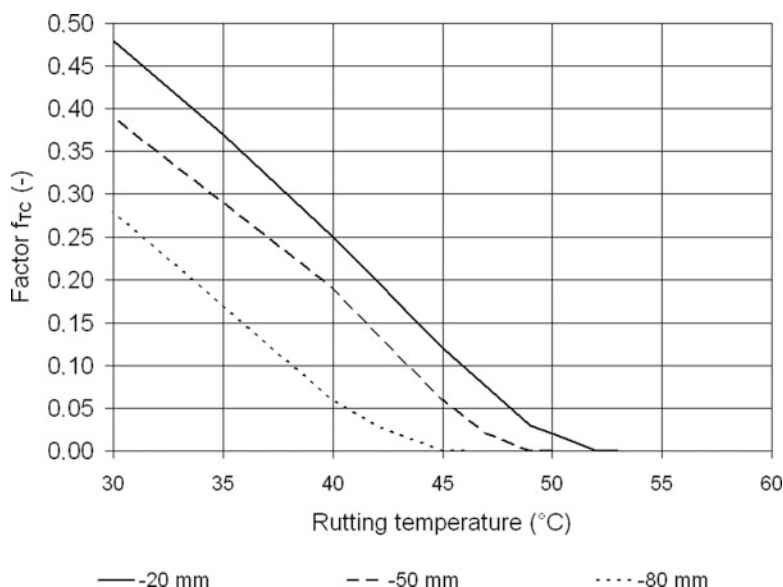


Fig. 5.17 Factor f_{TC} in various depth below the pavement surface and rutting temperature

The development of the rut depth in relation to the number of equivalent standard wheel loads of 50 kN at 45°C is shown in Fig. 5.18. The regression curve follows a power function with a regression coefficient of $R^2 = 0.966$.

$$RD = -0.7449 \times (ESWL_{45}'s)^{0.1536} \quad (5.17)$$

where:

RD = rut depth in mm

$ESWL_{45}'s$ = number of equivalent standard wheel loads of 50 kN and 45°C

A comparison with the predicted results was done by using the asphalt properties from Sect. 5.5.1.1. The stiffness modulus of the wearing course at 45°C for $f = 20$ Hz was calculated from the master curve with Eq. (5.7) and was found to be 704 MPa. With the help of Eq. (5.8) relative deformation and deformation rate (dynamic triaxial compression test) were calculated for the given properties (air voids = 2.3% by volume, traffic speed = 95 km/h) and the number of equivalent standard wheel loads.

As far as rut depth normalized to the course thickness is concerned, a reasonable agreement between the data collected in the field and the predicted data was found, as shown in Fig. 5.19.

Rutting in the field occurred slower than predicted. The expected rut depth in the wearing course was 6.2 mm whereas the value from field measurements was 5 mm.

Table 5.42 Development of average maximum rut depth (mm) in the right wheel track and number of standard equivalent wheel loads equivalent to 50 kN at 45°C

Year	Rut depth (mm)	ESWL ₄₅		
		Total SC	Total BC4	Lower part of BC4
1998	-4.42	88,109	15,949	3,632
1999	-4.53	127,904	21,019	4,900
2000	-4.71	210,292	31,356	5,755
2001	-5.07	293,848	46,990	6,242
2002	-5.32	382,514	68,823	10,724
2003	-5.63	508,729	113,052	17,403
2004	-5.62	562,559	126,547	17,496
2005	-5.90	651,285	150,520	21,141
2006	-6.09	751,364	165,773	22,496

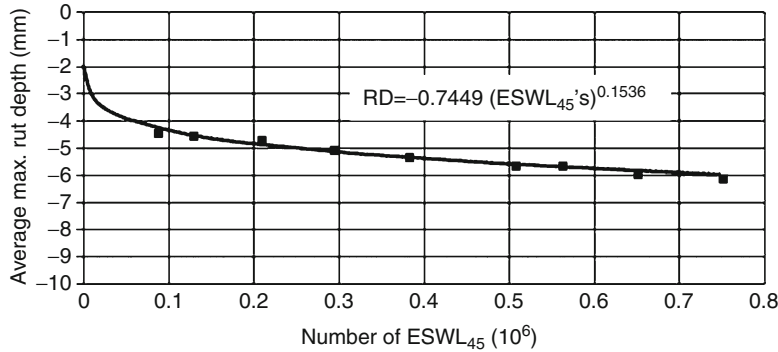


Fig. 5.18 Development of the rut depth in relation to the equivalent number of standard wheel loads of 50 kN at 45°C

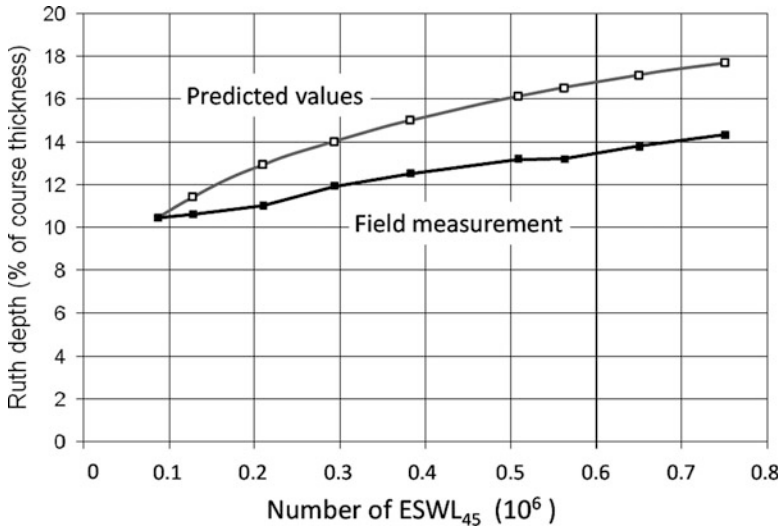


Fig. 5.19 Comparison between rut depth measurement in the wearing course and prediction in relation to the course thickness

Hence, the prediction was conservative and on the safe side in the sense that it was over-estimating rutting by about 25%.

5.6 Conclusions

This study on rutting behavior is based on interlaboratory tests on asphalt slabs cut from a new test section in Austria followed by an extensive 10 years survey of this test section with continuous traffic census and temperature measurements at different locations in the pavement as well as periodic rutting profile measurements. Long term traffic census and pavement measurements in the field were conducted deliberately not in a sophisticated but traditional way that is acceptable to a road operator at moderate costs and personnel resources.

The RILEM test section “A 10 Tauernautobahn” in Austria showed that in a middle European climate with high summer and low winter temperatures rutting proves to be one of the most likely damage cases for motorways.

The study also showed that rutting can be reasonably predicted in a conservative way with present tools and knowledge, if the temperature gradient and the distributions of wheel loads from the different types of vehicles together with tire type, tire pressure and tire configuration are taken into account accordingly. This result was achieved by following the checklist below:

- Conduct detailed continuous traffic census and climatic measurements.
- Prepare and evaluate in detail the basic data on traffic and climatic conditions.
- Define the ESWL (equivalent standard wheel loads) considering tire type, tire pressure, traffic speed for the project.
- Express the traffic data in ESWL's.
- Relate and compare daily air temperatures and asphalt pavement temperatures.
- Perform laboratory tests to determine stiffness moduli and the master curves in the hot period of all courses for knowing the ratio $E_1:E_2:E_3$.
- Determine the ratio of stiffness moduli for different temperatures in the various courses.
- Carry out wheel tracking or similar performance related tests on the upper part of the pavement at 30, 35, 40, 45 and 50°C.
- Determine the critical temperature where rutting occurs. (e.g. wheel tracking test after 50,000 loading deformation = 1 mm).
- Define a standard temperature, e.g. 45°C and rut depth.
- Calculate the critical ESWL's for the period of critical temperatures and for the intended period of prediction.
- And finally calculate with help of the Eq. (5.8) relative vertical deformation and deformation rate for the properties (air voids, traffic speed) and critical ESWL's.

However, although traffic and temperature data can be collected and their development can be predicted quite precisely, it seems difficult to perform damage predictions based on laboratory experiments and theoretical analysis as the variety

of test methods and prediction models appears quite large. Regarding the prediction of long term performance it seems important that uniform and standardized methods based on material characteristics and mechanical properties will be developed.

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Chapter 6

Mechanical Testing of Interlayer Bonding in Asphalt Pavements

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Abstract Steadily increasing requirements on pavement performance properties, in terms of bearing capacity and durability, as well as new innovative developments regarding pavement materials and construction, are observed worldwide. In this context interlayer bonding at the interfaces of multi-layered bituminous systems is recognized as a key issue for the evaluation of the effects, in terms of stress-strain distribution, produced by traffic loads in road pavements. For this reason a correct assessment of interlayer bonding is of primary importance, and research efforts should be addressed in order to improve the lack of correlation and/or harmonization among test methods. Following this principle RILEM TG 4 organized an interlaboratory test in order to compare the different test procedures to assess the interlayer bonding properties of asphalt pavement. The results of the experimental research are presented with a preliminary overview of basic elements, test methods and experimental investigations on interlayer bonding. Then the RILEM TG 4 experimental activities, based on the construction of three real- scale pavement sections, are presented in detail. Each pavement section was composed of two layers, and three different

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interface conditions were chosen. The first pavement was laid without interface treatment and the others with two different types of emulsion. Fourteen laboratories from 11 countries participated in this study and carried out shear or torque tests on 1,400 cores. The maximum shear or torque load and the corresponding displacement were measured, and the shear or torque strength was calculated as a function of the following parameters: diameter, test temperature, test speed, stress applied normal to the interface and age of the specimen. The results of this study are presented in terms of precision and correlations regarding the parameters which results in useful information on asphalt pavement interlayer bond tests.

Keywords Asphalt pavement • Interface • Interlayer bonding • Repeatability • Reproducibility • Shear strength

6.1 Overview on Interlayer Bonding in Asphalt Pavements

6.1.1 Introduction

A multi-layered pavement structure is usually composed of several distinct layers of different materials, laid on the subgrade. Normally, materials and thicknesses of flexible pavement layers are selected by the road management agencies considering economic availability, traffic loads and traffic composition, service life and climatic conditions in which the pavement has to work. This choice depends also on an adequate knowledge of material properties and interface characteristics. Many investigations have been carried out on materials, but not enough is known about the actual mechanical performance of interfaces.

The interface should prevent relative movements transferring horizontal shear stress between different layers. Among others, Peattie [1] and Kennedy et al. [2] showed that weak interlayer bonding or de-bonding, allowing these lateral movements, can produce unexpected and early failure of the pavement, affecting performance of multi-layered structure in different ways.

The likely causes and effects of poor interlayer bonding on multi-layered pavements have been studied in many research projects, and a summary of the obtained results is reported in the following paragraphs. In particular, the main aspects discussed in this chapter are:

- causes of poor bonding between different pavement layers;
- theoretical investigations on interlayer bonding;
- test methods to assess interlayer bonding;
- experimental investigations on interlayer bonding.

6.1.2 Causes of Poor Interlayer Bonding

The interface is not able to provide the required relative movement restraint because of a weak bond or de-bonding between the pavement layers.

The actual causes of poor bonding are not completely understood yet but it is possible to identify some factors that appear to play a significant role in increasing the risk of slippage failure due to de-bonding:

- type of base course material [1, 3]; it was shown that if the base course is constituted of specific types of materials (like wet-mix macadam or dry-bound granular material) and/or laid under unfavorable conditions (for example rain, inappropriate moisture content), the pavement structure is more subject to slippage failure;
- low compaction of base course or subbase course or subgrade which can cause low stiffness of the layer [1, 4–6], leading to material degradation at the interface, gradually resulting in a weak zone vulnerable to water damage, accelerating the loss of cohesion;
- segregation in the base course [5] due to aggregates of large nominal size (≈ 40 mm) and lack of bitumen. In particular, it could happen that the large size aggregates in the base course are poorly coated because most of the bitumen is used to cover fine filler. Because of segregation, such poorly coated aggregates are primarily located on cold material (subbase), producing a poor interlayer bonding;
- type of bitumen of the wearing course [1, 3, 4]; it is likely that lower viscosity and greater temperature susceptibility of the bitumen can cause the development of a slip plane at the interface if the mix is compacted at unnecessarily high temperatures;
- climate when the pavement is constructed [1–4, 6]; the gradient between the laying temperature of the wearing course and the temperature of the binder course has to be considered. In particular, a very hot wearing course material applied on the top of a cold binder course increases the risk of developing a plane with insufficient interlayer resistance;
- contamination of the lower layer surface [5, 6]; presence of dirt (dust or oil from construction traffic) on the top of the lower layer can result in reduced interlayer bonding;
- water flow between the layers [7, 8]; water flow in the pavement, due to surface cracking, produces a decrease in interlayer resistance as compared to ‘dry’ conditions;
- poor or excessive tack-coat application [4–6, 9]; lack of tack-coat (e.g. because of high aggregate absorption and/or porous surface absorption) may lead to weak bonding, whereas too much tack-coat could, at the same time, create a slip plane.

Clearly, this list is not exhaustive and many more reasons and their combinations may contribute to poor bonding between pavement layers. In addition, this list does not include the type of loading that is also likely to contribute to de-bonding and slippage failure.

6.1.3 Theoretical Investigations

To understand the role of the interfaces in a multi-layered bituminous system, for all possible pavement conditions, a theoretical stress-strain distribution evaluation is necessary.

Several computer programs exist that allow the modeling of pavement structures. Through proper material constitutive laws and proper pavement behaviour models, each program allows the assessment of stress-strain distribution of a loaded pavement system considering a different number of input parameters. In particular, stiffness modulus or thickness of the layer, number of layers, interlayer bonding level (expressed in terms of parameter K , explained later in this section), as well as vertical and horizontal loads transferred by traffic can be varied to observe their effects on the pavement structure.

To study pavement behaviour under traffic, many investigations are based on models that assume loads acting only in a vertical direction on the pavement surface [5, 6, 10–15]. These studies confirm that a proper knowledge of interlayer bonding conditions is imperative to accurately evaluate the stress-strain distribution in the pavement and thus its service life.

For this reason, a schematization of the interface conditions is necessary to study bonding properties. In particular, the interlayer mechanical behaviour has been modeled using the assumption that the interface can be described by elastic theory and considered as a thin material with shear modulus G and thickness h . The interlayer shear stress τ , due to pavement loading, causes a relative shear displacement $\Delta\xi$ between the two layers. This is expressed as:

$$\tau = G \cdot \frac{\Delta\xi}{h} = G \cdot \gamma \quad (6.1)$$

where γ represents the shear strain of the thin interface material. From Eq. (6.1), it is possible to obtain the well-known Goodman's constitutive law [16] describing the interface behaviour in multi-layered elastic systems, as follows:

$$\tau = K \cdot \Delta\xi \quad (6.2)$$

where K (equal to G/h) represents the interlayer shear stiffness typically expressed in MPa/mm.

In order to reduce numerical complications, in the computer implementation the following change of variable can be introduced:

$$K = \frac{l}{1-l} \quad 0 \leq l \leq 1 \quad (6.3)$$

where l represents a shear stiffness parameter. Then Eq. (6.2) can be rewritten as:

$$(1-l) \cdot \tau = l \cdot \Delta\xi \quad (6.4)$$

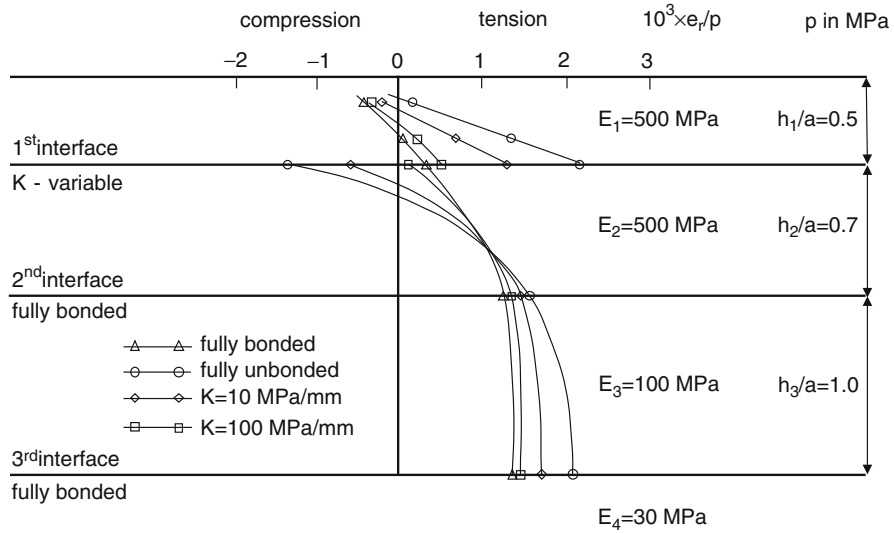


Fig. 6.1 Distribution of strain vs. depth [10]

Thus, in the case of fully bonded layers at the interface, the following relationships are valid:

$$K = \infty, l = 1, \text{ and } \Delta \xi = 0 \quad (6.5)$$

whereas, in the case of fully unbonded layers, the interface condition are represented in terms of:

$$K = 0, l = 0, \text{ and } \tau = 0 \quad (6.6)$$

The parameter K can be assumed as a characteristic value to measure the level of interlayer bonding and it has been investigated by several authors [10, 12–15, 17, 18] to evaluate the effects of different levels of bonding on pavement stress-strain distribution.

In particular, Uzan et al. [10] studied different K -values for the uppermost interface (from $K = 0 \rightarrow$ fully unbonded, to $K = \infty \rightarrow$ fully bonded) of a three-layered system on soft subbase. The authors found that the most considerable change in stress and strain values at the bottom of the upper layer occurred when K varied between 1 and 10^2 MPa/mm. Moreover, in the same study, it was shown that when the K -value increases, the strain approaches the value computed for a fully bonded interface. This increase in K -value, and thus in interlayer bonding, produces a decrease in the tensile strain at the bottom of each layer and an inversion in the strain sign at the top of the layers (Fig. 6.1). When a full bond between all the pavement layers is ensured, the maximum tensile strain is located at the bottom of the pavement. On the contrary, if one or more interfaces are not completely bonded,

as is normally the case, high tensile strains occur at the slipping interfaces. At the end, when complete de-bonding is achieved, the slippage at the interface occurs until the layers act independently.

Khweir et al. [5], Uzan et al. [10] and Shahin et al. [17] showed that even for bonding of 90%, behaviour very similar to the smooth condition can be found. In fact, Shahin et al. [17], investigating the effects of slippage at the interface of airfield pavement, found that when a very small amount of slippage takes place, the bottom of the overlay goes into tension horizontally and the strains in the pavement approach those of free slippage case.

The previous remarks concerning vertical forces acting on the pavement surface can be extended to horizontal forces that arise at the contact zone between vehicle tire and pavement, in braking or accelerating road sections (e.g. intersections areas, steep ramps) or in sharp curves. Different theoretical modeling efforts [17–21] showed that adding horizontal loads to normal forces makes the stress-strain conditions in the pavement more severe.

As far as strain distribution is concerned, Shahin et al. [17] and Romanoschi and Metcalf [19] showed that when horizontal loading is present, large tensile strains develop at the top of the surface layer, implying that a crack may initiate at the pavement surface (top-down cracking) and not at the bottom of the layers (bottom-up cracking), as is usually assumed in current mechanistic design procedures. In fact, Myers et al. [22] and Myers and Roque [23, 24] showed that the primary contributors to surface-initiated longitudinal wheel-path cracking are transverse contact stresses induced by tires. Hence, when horizontal forces act on the top of the structure, the strains originated in correspondence of the surface reduce the pavement service life [17, 19, 20]. This reduction in pavement service life, due to failure of the surface course, also arises when a certain level of de-bonding occurs at the uppermost interface, both with [17–19, 25] and without [5, 11, 13, 17, 25, 26] horizontal loads. In particular, Romanoschi and Metcalf [19] showed that the cumulative effect of interface de-bonding and horizontal forces acting on the pavement surface may lead to a reduction of pavement life by up to 300 times for semi-rigid pavements and 15 times for flexible pavements.

In terms of stress distribution, Hachiya and Sato [18] and Kulkarni [21] showed that the shear stress at the uppermost interface becomes larger if horizontal surface forces are considered, increasing its value when the magnitude of horizontal loads increases. Moreover, it has been shown that an increase in layer thickness reduces the interlayer shear stress also due to the higher distance between the interface and the pavement surface.

As mentioned, the interlayer bonding condition affects pavement bearing capacity and performance. Literature surveys agree in considering partial bonding as the best representation of the in situ interlayer condition. In particular, Peattie [1] reports an investigation, carried out in 1966, according to which 55% of specimens taken from un-trafficked roads, in new-constructed pavements, have poor interlayer bonding. However, several studies [2, 27, 28] experimentally showed that a form of re-bonding between bituminous layers may occur under traffic during pavement life, in particular under the wheel tracks.

From a theoretical point of view, the mechanical behaviour of the interfaces is closely connected to the stiffness moduli of the pavement layers. The stiffness assessment can be performed through the back-calculation analysis that uses the deflection basin obtained by FWD (Falling Weigh Deflectometer) measurements. A practical limitation of most back-calculation methods is assuming a complete bond between pavement layers, which does not represent the actual in situ condition. Obviously, significant errors in back-calculated layers moduli arise from this unsuitable modeling of interlayer bonding condition of pavement structure interfaces [6, 12–15]. In particular, Al Hakim et al. [6, 12] and Romanoschi and Metcalf [14] agree that the back-calculated modulus of the subgrade is not much influenced by the errors generated from an incorrect modeling of the interlayer bonding condition. As far as the other back-calculated moduli are concerned, it has been shown that, assuming perfect-bonding, bituminous layer stiffnesses found with the FWD test are usually lower than the real ones.

To overcome this modeling limitation, an advanced back-calculation method was proposed by Al Hakim et al. [12], Al Hakim [15] and Al Nageim and Al Hakim [13]. This method was applied to a wide range of structural pavements that were representative of in situ conditions in terms of materials and/or thermal variations. The method assesses the order of magnitude of the layer stiffness moduli considering the actual interlayer bonding condition by using the forementioned interlayer shear stiffness K . Two limiting values for K were identified: K below 10^{-2} MPa/mm represents the perfect slippage condition (complete de-bonding), whereas K above 10^2 MPa/mm means perfect bonding. The real condition of interlayer bonding should be in the range between these two limits.

The knowledge of the actual K -value is thus necessary to correctly evaluate the pavement bearing capacity and the layer moduli by using the available procedures for rational design and maintenance of flexible pavements. Therefore, several test methods have been designed in order to evaluate the most appropriate K -values considering different test configurations and different test devices that produce different qualitative results.

6.1.4 Test Methods

Considering the fundamental importance of a correct assessment of interlayer bonding condition and the increasing interest in this matter, the lack of correlation and/or harmonization among test methods to characterize interlayer behaviour is surprising.

During recent years, many countries have developed methods and equipment for testing interlayer bond of pavement layers [29]. Despite these efforts, no internationally recognized test method or procedure has been established so far. In fact, test methods and procedures still vary regarding test devices, loading conditions, specimen geometry and preparation as well as test temperature [30].

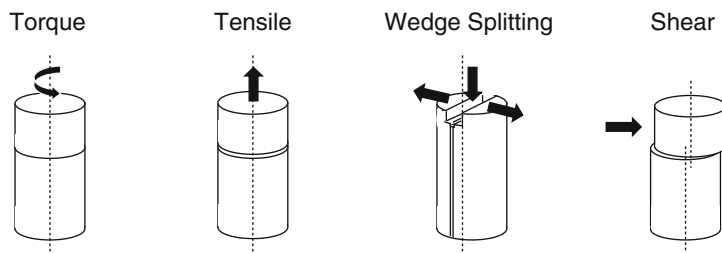


Fig. 6.2 Working schemes of destructive interlayer tests

Due to this diversity, test results from different countries and laboratories are often not comparable.

The recognized test methods can be divided into two categories:

- Destructive tests;
- Non-destructive tests (NDT) on existing pavements.

The following summary takes into consideration the test devices currently used to evaluate interlayer bonding. In particular, it is worth noting that since different test conditions and configurations are considered for different test equipment, the results obtained are usually non-uniform and non-homogeneous.

6.1.4.1 Destructive Tests

Most destructive methods require laboratory testing of specimens where at least two layers from a pavement structure or laboratory specimen are considered. The specimen is brought to failure at a constant loading or displacement rate depending on the specific test device. Some drawbacks are linked with this procedure in the case of core sampling: it is destructive for a part of the pavement, time consuming and causes traffic delays.

Based on test configurations or loading procedures, the destructive interlayer test methods are grouped into four categories (Fig. 6.2):

- torque tests;
- tensile tests (pull-off tests);
- wedge splitting tests;
- shear tests (with or without normal load).

Each laboratory optimized its own test method. The following paragraphs contain a summary of the experimental devices currently used in the world. Note, that fracture mechanical tests for testing specimens with notches at the interface are not considered here. This is also true for bending configurations, since this type of test is of limited suitability for interlayer bond investigation of pavement layers.

Torque Test

The interlayer torque test method involves the measurement of the peak shearing torque, applied on cored specimens, to cause the de-bonding of a surface system from its substrate. In particular, a torque is applied to the top of the specimen, which induces a twisting failure mechanism in the bond layer.

A commonly available hand held torque wrench is the basis of the test performed in the United Kingdom and it can be carried out on both 100 and 150 mm diameter specimens.

This method allows the comparison between in-situ and laboratory prepared specimens. The in-situ specimen is cored just below the upper layer in order to ensure that the failure occurs at the interface, whereas the laboratory specimen is cut and extracted, taking care not to damage the surface layer or the interface of the core. For the laboratory testing, double-layered cores are clamped or glued to a firm base. In both cases a steel plate is glued on top of the specimen acting as an adapter between core and torque wrench, which is then attached to the plate. A torque is applied until failure, and the bond strength is calculated using the measured maximum torque moment at failure.

It is worth noting that the stress distribution at the interface between the two layers is non-uniform in this type of test because the shear stress varies from zero at the centre to a maximum value at the outside of the core. Hence, failure is non-uniform and initiates at the outside and propagates towards the centre of the core.

Some care should be taken performing this test:

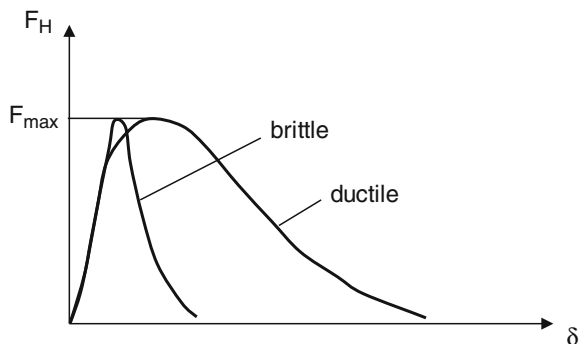
- the adhesive to fix the metal plate to the upper part of the core should be strong enough to avoid failure within the adhesive;
- a small direct compression load, normal to the plane of induced shear, should be applied to mitigate the possible lifting or compressive force on the core due to difficulties in applying the force parallel to the bond surface.

Moreover, an important limitation of this test consists of its unfeasibility if the interlayer bonding resistance is higher than the torsion resistance of the material of the single layers.

Tensile Test (Pull-Off Test)

The interlayer pull-off test can be carried out both in situ and in laboratory. This rather simple test consists of applying a tensile stress to a double-layered specimen. Its top surface is glued to a steel plate that is pulled off in axial direction up to the breaking of the interface. The obtained result is the maximum load registered during the test. In laboratory tests, the bottom surface is glued to a fixed steel plate.

Fig. 6.3 Load-displacement curves for brittle and ductile materials



This test method has some limitations that are summarized in the following:

- possible eccentric load transmission due to an inclined testing piston;
- stress concentrations due to constrained transverse strains in the area of load application owing to the presence of the glued plates.

Moreover, the two most important limitations are the unfeasibility of this test if the interlayer bonding resistance is higher than the tensile resistance of the material of the single layers (as already observed for torque test) and the impossibility of evaluating the grain interlock produced during the compaction, so no dilatancy can be studied.

Wedge Splitting Test

The working scheme of the interlayer wedge splitting test, developed by Tschegg et al. in 1995 [31], consists of opening a crack in a double-layered specimen until a complete separation of the two layers takes place. To perform this test, specimens prepared in different shapes [31] and produced in laboratory or cores cut from road pavements have to be used.

The obtained result is a load-displacement (F_H - δ) curve that allows the evaluation of the fracture mechanical properties, such as brittle or ductile behaviour, of heterogeneous materials (Fig. 6.3). The test shows two not negligible disadvantages: the specimen stress distribution is not representative of the field conditions associated with slippage and the specimen preparation requires considerable efforts due to the complicated specimen shape.

A similar test, inspired by Tschegg et al. [31], has been proposed by Bourdrel and Chabert [32]. The test consists of a wedge advancing into a double-layered specimen at a rate of 2 mm/min, and the obtained result is a force vs. time curve.

Shear Tests

Currently, the most popular test configuration for laboratory assessment of bonding condition is based on interlayer direct shear testing. A constant rate of shear

displacement (or a constant rate of shear loading) is applied at the interface of double-layered specimens until interface failure occurs. The typical results of this type of test are characterized by shear strength.

Many different types of shear test equipment have been developed in different countries and they can be classified into two categories: pure direct shear tests (without normal stress) and direct shear tests with normal stress applied.

Pure Direct Shear Tests

The most common pure interlayer direct shear equipment is used in the shear test designed by Leutner in 1979 [33]. The test consists of applying a shear displacement, at a constant rate, across a pre-defined plane monitoring the resulting shear force and the applied displacement. It provides the shear force-shear displacement curve at the investigated plane. The maximum shear stress, obtained at failure condition, is easily computed considering the cross sectional area at the interface of double-layered specimens.

The Leutner shear test is used in many countries [28, 33–35] because its arrangement is very simple and it is a relative rapid test for investigating interlayer bonding of pavement layers. The testing frame is fitted into an ordinary testing machine (Fig. 6.4) and allows testing of specimens with a diameter of 100 or 150 mm, taken either from a pavement structure or produced in the laboratory. The displacement rate is normally 50 mm/min as for Marshall test. However, there are some limitations of the original test device:

- no gap between the shear rings at the interface, so that interlayer shear stress and shear stiffness may be influenced by the aggregates of the two mixes; in order to solve this problem a gap of 5 mm has been introduced in the United Kingdom [36, 37]; however a specific study showed that the influence of the gap width depends on the asphalt concrete type of the two asphalt layers, especially when the material characteristics of the mixtures are different [38];
- unfeasibility to apply a force perpendicular to the interface to schematize better the traffic load condition;
- non-uniform distribution of interface stresses;
- possible load eccentricity that causes an additional momentum.

A modified version of the Leutner shear test equipment is the LPDS (Layer-Parallel Direct Shear) test device, set up by Swiss Federal Laboratories for Materials Science and Technology – EMPA [7, 8, 27, 39, 40]. This device (Fig. 6.5) is very similar to the Leutner test but some differences can be identified:

- gap width of 2 mm between the yoke and the pneumatic clamp;
- specimen clamping: one part of the core is laid on a circular u-bearing and held with a well-defined pressure by a semicircular pneumatic clamp;
- adjustable yoke and clamp to accommodate specimens with different diameters (from 148 to 155 mm) or prismatic specimens with width of 150 mm and height of 130 mm.

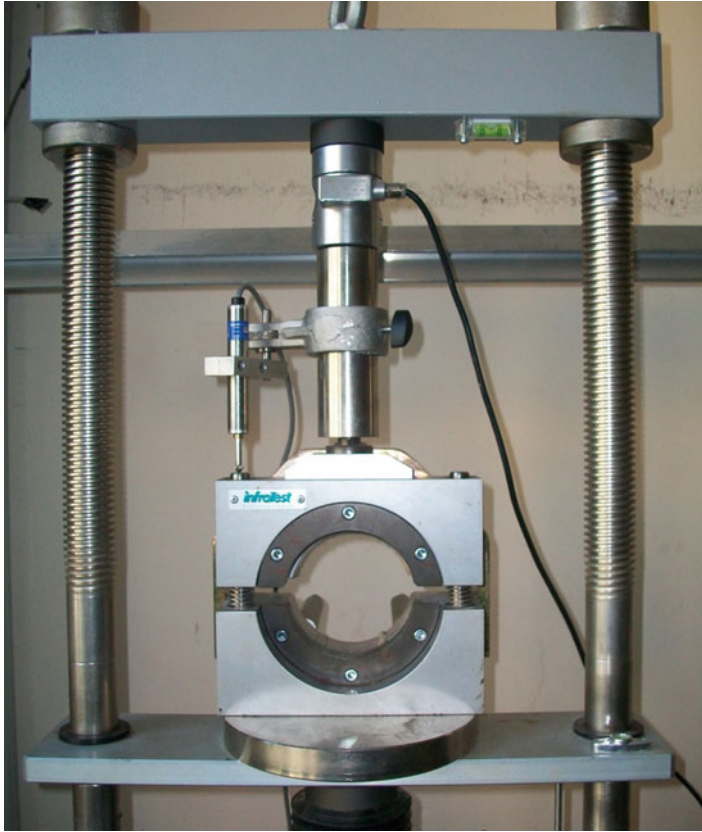


Fig. 6.4 Setup of the Leutner shear tester

Since 2000, the Leutner test is incorporated into the Swiss Standard SN 671961 [41].

Similar equipment has been used by Molenaar and Heerkens [42]. The shear test device holds the bottom part of the double-layered cylindrical specimen and a shear load is applied at the interface. A Marshall stability loading press is used to apply a constant displacement rate of 51 mm/min. The test is performed on 101.4 mm diameter specimens. The obtained result is a shear force-shear displacement curve that allows the calculation of the maximum shear stress and of the shear stiffness K .

A test device comparable to the previous equipment is the Shearing Apparatus designed and constructed at the Florida Department of Transportation (FDOT) and described by Sholar et al. [43]. The device is mounted on a Material Testing System (MTS) that allows the temperature to be controlled. The shear force of double-layered specimens of 150 mm nominal diameter, cored from the pavement or prepared in laboratory, is measured at a constant displacement rate of 50.8 mm/min. This equipment is manufactured with a gap of 4.8 mm between the shearing

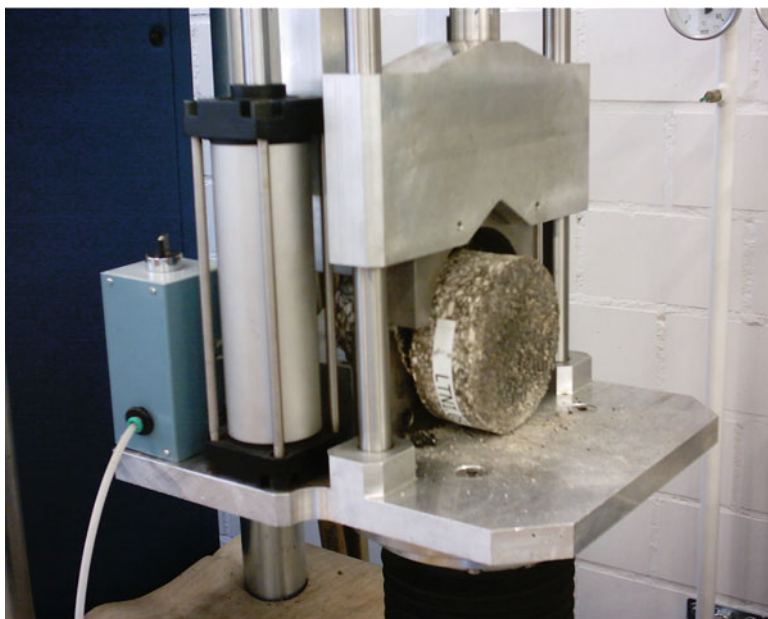


Fig. 6.5 Setup of the LPDS test device [7]

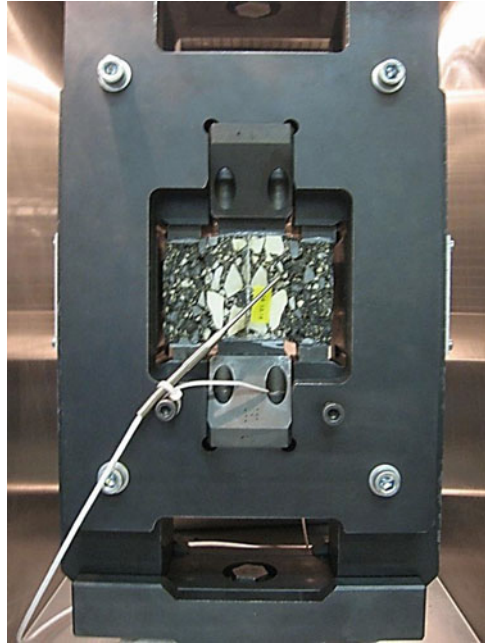
platens to accommodate cores not extracted exactly perpendicular to the pavement surface.

The maximum shear force at failure is recorded through data acquisition software and it allows the shear stress at failure to be obtained.

A modified version of the test apparatus from ASTM D-143, which is used to test wood specimens, has been employed by Mrawira and Damude [44]. The device is of the guillotine style and can apply a uniform lateral load through a pivoting load surface. The gap between the shearing planes is 8 mm to account for the not perfect parallelism of the layers. Specimens with a diameter of 102 mm are tested at a constant displacement rate of 1 mm/min. The testing machine automatically records the shear force and the shear displacement allowing the maximum shear stress to be calculated.

A pure direct shear test configuration different from the previous devices, the LCB shear test (Laboratorio de Caminos de Barcelona), has been developed at the Technical University of Catalonia [45–47]. In the LCB test working scheme a cylindrical specimen (with diameter of 101.6 mm) is used as a beam located over two supports. The test section is very close (5 mm) to the support so that the bending moment is almost zero and the specimen is effectively subjected to the shear stress alone. A loading piston is located over the metallic mould and a constant deformation rate of 1.27 mm/min is applied to the specimen. The load-deformation data allows the maximum shear stress to be calculated knowing the cross-sectional area of the specimen.

Fig. 6.6 Direct shear test with instrumented specimen



A shear interface fatigue test (Fig. 6.6) has been developed at University of Limoges: the Double Shear Test (DST) [48, 49]. It allows the investigation of asphalt concrete prismatic specimen behaviour in shear monotonic and fatigue loading, under controlled temperature conditions. The shear loading is applied through a servohydraulic machine, allowing controlled displacement or controlled force. The specimen ($70 \times 120 \times 50 \text{ mm}^3$) is composed of three parts (layers), two by two separated. In the device working scheme the lateral parts are fully fixed, whereas the central part is subjected to a displacement. Six steel plates are glued onto the specimen and screwed with the device. The gap between plates can be adapted from 4 to 10 mm. Homogeneity of the shear stress distribution was optimized by FEM modeling and verified by digital image analysis. The shear displacement at the interfaces is measured by an extensometer. Temperature and force and displacement are recorded through a data acquisition system and the obtained result is a shear force-displacement curve, related to time and number of cycles.

Direct Shear Tests with Normal Stress Applied

Uzan et al. [10] designed a direct shear box to evaluate the shear stiffness K for different test conditions. This equipment allows the application, together with the shear force, of a load perpendicular to the interface of double-layered prismatic specimens, prepared in laboratory. The test is performed at a constant shear displacement rate of 2.5 mm/min. During the shear process the application of a

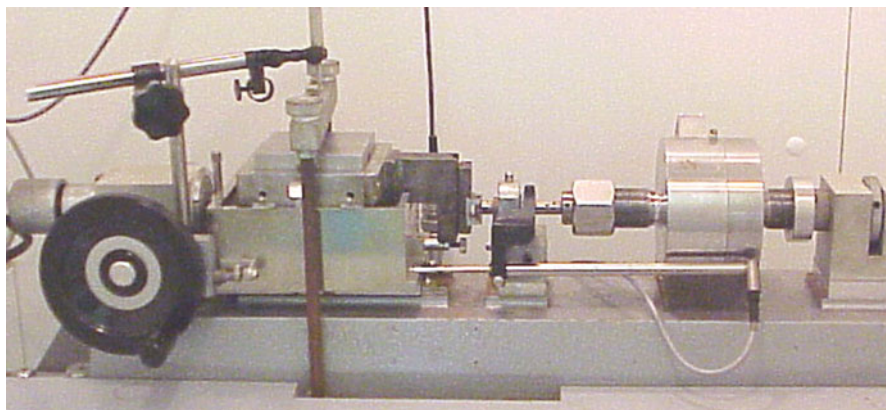


Fig. 6.7 ASTRA shear test device

shear force produces a distortion in the two parts of the specimen and the increase of a non-homogeneous state of stress. In particular, a recent FEM analysis [50] showed that, at the beginning of the test, as the distance from the shear load application increases, the shear stress along the interface decreases causing a concentration of shear stress in the vicinity of the load application. On the contrary, with the advancing of the test, when slippage occurs, the shear stress distribution is almost uniform along the interface, allowing the calculation of the maximum shear stress as ratio between the maximum shear force and the area of the interface. The same equipment has also been used by Ishai and Livneh [51] to investigate the adhesion properties of prime coats spread at the interface between unbound base layers and asphalt courses.

Conceptually similar equipment, the Dynamic Shear Box, has been developed at the University of Nottingham by Brown and Broderick to investigate reinforced asphalt. This device is also used to assess interlayer bonding between pavement layers. It allows testing of specimens of $320 \times 200 \times 120 \text{ mm}^3$ dimensions, which are subjected to repeated shear loads while a normal load is also applied. Kruncheva et al. [50] showed that the disadvantage of this equipment is the non-uniform interface stress distribution that needs proper assumptions for the obtained results [50, 52].

At Delft University, de Bondt [53] developed the Four Point Shear Test based on the use of two hydraulic actuators to apply shear and normal loads at the interface of specimen layers. In this test configuration no bending moment is generated along the shear plane because of the position of the supports, allowing a uniform shear stress distribution at the interface to be obtained (verified by the FE model). However, due to its complexity it is only used for research purposes.

The ASTRA (Ancona Shear Testing Research and Analysis) apparatus (Fig. 6.7), designed at Università Politecnica delle Marche [40, 54–57] is a direct shear box. It allows the evaluation of multi-layered bituminous systems performance under laboratory conditions, testing both prismatic (cross sectional area of

100 × 100 mm²) and cylindrical specimens (diameters from 94 to 100 mm). The specimen is installed in two half-boxes separated by an unconfined interlayer shear zone. A constant vertical load, perpendicular to the interface plane, is applied during the test carried out at a constant displacement rate of 2.5 mm/min. The whole apparatus is located in a climatic chamber with temperature and relative humidity control. ASTRA interlayer shear test returns a data set that allows the calculation of the maximum shear stress at failure and the dilatancy that is a geometrical parameter. Since 2007, the ASTRA test is incorporated into the Italian Standard UNI/TS 11214 [58].

Shear tests have been conducted with the Superpave Shear Testing (SST) machine by Mohammad et al. [59, 60]. The shearing apparatus is mounted inside the SST chamber and the specimen is subjected to a constant load rate of 222.5 N/min until failure. The result is a shear force-displacement curve, from which the maximum shear stress is obtainable. It is worth noting that stress-controlled tests are not appropriate to investigate the post-failure behaviour at the interface. Mohammad et al. [9] further developed the Louisiana Interlayer Shear Strength Tester (LISST), new testing equipment that can be placed into any universal testing machine. A cylindrical specimen is placed among a shearing and a reaction frame and, during the test, vertical and horizontal displacements are measured through sensors. The device allows the application of a shearing load up to failure in the presence of a constant normal load on specimens with either 100 or 150 mm diameter.

Another type of direct shear test device has been proposed by Romanoschi and Metcalf [20, 61]. This apparatus permits the application of both shear and normal force at the interface of double layered samples. The test specimens can be extracted from the pavement or prepared in laboratory with a diameter of 95 mm. During the test, carried out at a constant displacement rate of 12 mm/min, the normal force, the shear force and the displacement are measured and controlled. The test result is a shear force-shear displacement curve that allows the shear stress-shear displacement curve to be obtained considering the cross sectional area of the specimen.

Since the pavement structure is subjected to a repetitive mechanical action transferred by moving vehicles, Romanoschi and Metcalf [61] also proposed a shear fatigue test obtained by modifying the described equipment. In particular, the longitudinal axis of the specimen is at 25.5° with the vertical so that the applied interface shear stress is about half of the normal stress. The cyclic vertical force is provided by a hydraulic frame with 5 Hz frequency. The test provides a set of parameters that characterize the fatigue shear behaviour of the interface.

A different type of shear test device has been designed [62] at the National Centre for Asphalt Technology (NCAT) of Auburn University. This equipment allows the application of a load perpendicular to the interface together with a shear load, both applied with a Marshall press frame. The diameter of the test specimen is 150 mm and the constant displacement rate of the test is 50.8 mm/min. The load and deformation data are recorded in a digital format allowing the shear force-shear displacement curve and the maximum shear stress to be obtained.

6.1.4.2 Non-destructive Tests

Along with the destructive tests, a range of non-destructive techniques (NDT) can be successfully applied to investigate interlayer bonding. They are directly performed in the field without destroying the pavement layers.

These techniques are generally based on the propagation of waves of frequencies higher than 20 Hz (limit of human audible frequency), into the pavement structure. The procedures are different in terms of the method used for generating the wave impulse and the acquisition system.

The most common techniques are:

- Portable Seismic Pavement Analyser (PSPA);
- Colibrì test;
- Hammer test;
- Falling Weight Deflectometer (FWD).

Nazarian et al. [63] developed the Seismic Pavement Analyser (SPA) that consists basically of a wave source and of a specific receiver that picks up the returning waves. It can be used for many different purposes, and one of these is the evaluation of interlayer bonding of pavement structures. This device implements five “seismic” testing methods (described by Nazarian et al. [62]), is computer controlled and the data collection is performed automatically.

The same technology has been implemented in a smaller and less expensive device called a Portable Seismic Pavement Analyser (PSPA) that may be used for determining the modulus and the thickness of the upper pavement layer and for an approximate assessment of the bonding condition. Details about the applicability of the PSPA for bond measurement are given by Kruntcheva et al. [64].

At the beginning of the 1990s, the French National Public Works Laboratory developed a non-destructive testing device, called Colibrì [65]. In this test, a mechanical impulse is applied to the surface of the pavement and its response is measured by a set of accelerometers. A spectral analysis of the signal makes it possible to assess various aspects of the pavement structural condition including de-bonding between layers.

The Hammer test has been designed and developed at University of Nottingham [66, 67] and it allows the evaluation of the in-situ bonding condition using an impulse hammer approach. It involves the measurement of the vertical acceleration of a pavement area adjacent to a point where an impulse is applied by a hand-held impact hammer. Assuming that the loss of adhesion between pavement layers is reflected in the dynamic response of the pavement structure, the shape of the signal obtained from the site is analysed and used to evaluate bonding condition.

Due to complexities associated with data interpretation, it is unlikely that this test will be able to be used to quantitatively determine in-situ bond condition but, nonetheless, it has potential for use as a rough qualitative evaluation tool that could be used to indicate where cores should be taken for detailed laboratory testing.

The same can be asserted also for FWD measurements. In particular, interlayer bonding tests are usually required to support the pavement evaluation findings

obtained with FWD. In order to simulate the traffic action, this equipment applies an impact load to the pavement surface allowing the deflection basin to be obtained. It is then used in the back-analysis process that consists of optimising the set of pavement parameters (layers stiffness moduli) to fit in the best way possible the measured deflections.

Al Hakim et al. [6] and Al Hakim [15] developed and validated a new back-calculation analysis procedure for the assessment of bonding condition between pavement layers using FWD test results. In particular, in addition to the layers stiffnesses, the new back-calculation analysis process is performed to calculate the shear stiffness K between the bituminous layers which gives an indication of the in-situ interlayer bonding condition.

A further study on the evaluation of interlayer bonding by means of FWD tests was also performed by Mehta [68], demonstrating that when the stiffness ratio between the top layer and the second layer of a flexible pavement is greater than 10 (and if the second layer stiffness is greater than a minimum value), the pavement can be considered less affected by slippage compared to the case characterized by a stiffness ratio less than 10.

6.1.5 Experimental Investigations

The knowledge of the influence on interlayer bonding of different parameters, such as material characteristics, interface properties, field related testing parameters and specimen conditions allows a rough assessment of pavement structure performance. Many experimental investigations have been performed by several authors to obtain information about the factors that negatively affect bonding conditions and thus stress-strain distribution.

6.1.5.1 Materials Characteristics

Previous experimental research has shown that, in particular, two material characteristics have to be taken into account in the investigation of interlayer bonding: mix grading of the two layers in contact and their air void content.

As far as *mix grading* of layers is concerned, Sholar et al. [43] and West et al. [62] found apparently contradictory results comparing fine-graded mixes with coarse-graded mixes. However, several authors [34, 40, 43, 62] agree that a correct mixture design is fundamental to ensure good interlock at the interface. In this sense, Raab et al. [69] evaluated the influence of aggregate interlock, combining different aggregate sizes, on the maximum shear stress between two layers using a model material composed of idealized aggregates represented by steel balls and 10/20 penetration graded bitumen. Shear bond testing was done using the Layer Parallel Shear Test (LPDS) with two different test speeds as well as a Shear Box with varying normal stress. This study showed the influence of the ball size and/or

the combination of different ball sizes on the maximum interlayer shear stress (e.g. shear strength) and shear stiffness for both test speeds.

Several investigations [27, 28, 34, 39, 47, 52, 70] showed that the interlayer resistance of the uppermost interface is strongly influenced by the *air void content* of the surface course. In particular, all the aforementioned research agrees that a well-compacted upper asphalt layer leads to higher maximum interlayer resistance and, accordingly, inadequately compacted specimens display results lower than those for normally compacted material. In this sense, Santagata et al. [71] investigated air void content at the interface by using X-ray Computer Tomography and showed that both lower and upper layer air voids affect interlayer shear resistance in a non-proportional way. Moreover, the study showed that air void content values obtained with traditional methods are suitable for studying the influence of air voids on interlayer shear resistance.

Several authors [27, 31, 35, 59, 60] performed a comparison between shear properties of monolithic specimens and interlayer shear resistance of double-layered specimens. All these studies showed that the construction of flexible pavements in multiple layers introduces weak zones at the interfaces. However, it has been shown that the interlayer shear resistance of specimens, both without interface treatments (Partl et al. [27]) and with tack-coat (Collop et al. [35]), approaches the values obtained from tests on monolithic specimens.

6.1.5.2 Interface Properties

As far as interface bonding properties are concerned, different influence parameters must be taken into account:

- **Tack-coat application.** To guarantee good bonding during pavement construction or placing of overlay, tack-coats (also known as bond coats) are usually applied at the various interfaces. These tack-coats are typically water-based bitumen emulsions, unmodified or polymer modified, but performance-graded asphalt binders are also used in some cases. Several authors investigated the effectiveness of tack-coat comparing different interface treatments taking also into account non tack-coated interfaces.

It has been experimentally shown that interlayer bonding significantly increases if modified emulsions were used as tack-coat in comparison with both non-modified emulsions [18, 26, 40, 46, 47, 72–74] and non-tack-coated interfaces [26, 40, 72, 74].

Many researchers [9, 21, 26, 52, 54–57, 72, 75] agree that non-modified emulsions spread as tack-coat are beneficial to interlayer bonding as compared to untreated interfaces. Hachiya and Sato [18], Raab et al. [39] and Mrawira and Damude [44] do not agree with this statement, showing that an improvement of adhesion properties can generally not be observed when the maximum interlayer shear force for different tack-coat types is compared with the uncoated condition. In fact, they found that the difference in interlayer stresses

between tack-coated and non-tack-coated interfaces is not significant in comparison with the variation observed between different specimens with the same interface treatment. Moreover, Mrawira and Damude [44] and Canestrari et al. [40] showed that non-tack-coated interfaces seem to exhibit slightly higher maximum interlayer stress than these with non-modified emulsions. This behaviour could be due to a high amount of applied emulsion and/or a low mechanical performance of the residual bitumen (e.g. emulsion type as verified also by Mohammad et al. [74]). Hence, it is not possible to assert with adequate confidence if non-modified tack-coats provide an increase in interlayer bonding.

As far as the performance-graded asphalt binders are concerned, West et al. [62] showed that modified bitumen emulsions produce higher interlayer bonding than bitumen emulsions, whereas Kulkarni [21] and Mohammad et al. [60] found that both treatments have comparable maximum interlayer stresses. Also, in this case, the different results could be due to the different amount of applied bitumen products, the specimen preparation technique and/or the different mechanical performance of the residual bitumen.

These results explain why one of the greatest problems in designing a pavement structure is the choice of the correct quality and quantity of tack-coat to be applied at the interface. In fact, too much tack-coat may weaken the interface by introducing a slip plane instead of mobilising a bond between the layers. Many authors [10, 18, 43, 56, 59, 60, 62, 72, 74] confirm that there is an optimum amount of emulsion that depends not only on bitumen, mixes and pavement characteristics [40] but also on temperature [40, 59, 60] and other parameters. However, it could happen that the increasing of the residual rate of tack-coat does not significantly change the bond strength at the interface, as observed by Tashman et al. [76].

Regarding the mix characteristics, Mrawira and Damude [44] and West et al. [62] showed that the tack-coat amount depends on the state of micro and macro texture of the lower layer surface.

As far as the pavement characteristics are concerned, several authors experimentally confirmed what some international recommendations suggested about the correct tack-coat amount to use:

- Jain et al. [77] reported about the Indian usual procedure to select the amount of tack-coat in function of existing pavement surface conditions in terms of bitumen content;
- similarly, Hachiya and Sato [18], according to the Federal Aviation Administration [78], confirmed that the amount of tack-coat must be chosen carefully, taking into account the condition of the existing pavement surface, both for airport pavements and road constructions;
- according to British Standards [79] the amount of tack-coat or bond coat (emulsion with modified bitumen) is a function of the bitumen content of the asphalt layers for new constructions and of the existing pavement conditions for maintenance intervention;

- **Lower layer surface roughness condition.** Some studies [44, 71] showed that the surface micro and macro texture of the lower layer seems to play a key role for the interlayer bonding. Regarding this issue, Santagata et al. [71] showed that lower roughness means lower interlayer shear resistance. Similarly, Sholar et al. [43], West et al. [62] and Tashman et al. [76] found that for equivalent test section conditions, the interlayer resistance for milled surfaces is significantly higher than for non-milled surfaces due to the rough longitudinal grooving in the surface created by the milling machine;
- **Lower layer surface contaminations** (debris, dirt, dampness, oil). In this case, several researchers [39, 52, 66] demonstrated that the presence of dirt particles at the interface negatively affects interlayer bonding and decreases the maximum interlayer stress as compared to a clean interface. Several authors [18, 28, 35] showed that this decrease is more evident if a tack-coat is spread on a contaminated interface. On the contrary, Bocci and Canestrari [56] and Raab and Partl [80] showed that in the case of a dirty base layer surface the advantage of the application of a tack coat is clearly revealed, because its use would restore the bond between the layers leading to acceptable interlayer bonding.
- **Interface moisture.** Sholar et al. [43] found that interlayer bonding of wet sections may also increase with time, but not to the same magnitude as the equivalent sections without presence of water.
Regarding the combined effect of wet interface and tack-coats, Raab and Partl [39] showed that, in the case of a wet surface, the use of tack-coats can lead to better adhesion properties, whereas other research [43] found that placing hot mix asphalt over a tack-coat material that has been contaminated by rain is potentially detrimental to the performance of the hot mix asphalt, possibly resulting in immediate or future slippage or delamination. This disagreement between different authors could be due to the differences in water content of the used tack-coat or to the residual bitumen properties;
- **Immersion in water.** Raab and Partl [7] and Raab [8] showed that the presence of water produces a decrease of 15–27% in interlayer resistance when compared with the test results of specimen without water immersion. This difference becomes negligible if a tack-coat is spread at the interface [39, 46].

6.1.5.3 Field Related Testing Parameters

Testing temperature is one of the most important influence parameters because of the bitumen temperature dependence.

Many authors [10, 18, 21, 27, 31, 40, 47, 57, 59, 61, 62, 74, 81] agree that both interlayer forces and stiffness values are extremely temperature dependent for any interface treatment (with or without tack-coat). In particular, for temperatures higher than 0 °C, one finds that the lower the test temperature, the higher the interlayer resistance.

Raab [8] found that very low temperatures (−20 °C) may be critical for the interlayer bond due to weakening of the pavement structure resulting in a decrease

of the bond properties. Tschegg et al. [31] and Mirò Recasens et al. [46] showed that, at temperatures below 0 °C and in presence of tack-coat, the trend observed for higher temperature is subjected to a stop and, sometimes, to a reverse, such that lower test temperatures result in lower interlayer resistance. Mirò Recasens et al. [46] supposed that this interlayer bonding behaviour could depend on the Fraass brittle point of the emulsion bitumen.

To solve the problem of the interlayer behaviour at low temperatures, Canestrari and Santagata [57] compared interfaces with and without treatment showing that the effectiveness of tack-coats becomes negligible below 10 °C. For this reason, the study of the effects of very low temperatures on interlayer resistance becomes not relevant for tack-coat interface treatments.

Traffic conditions in terms of vertical load applied on the surface significantly influence interlayer bonding.

Several authors [9, 10, 40, 55–57, 59, 62] showed that increasing normal forces generally increase interlayer bonding resistance, at any given test temperature and for any interface treatment condition (with or without tack-coat). Romanoschi and Metcalf [14, 19, 20, 61] partially disagree with the previous authors, stating that the maximum interlayer resistance is not affected by the normal stress level in the presence of a tack-coat but is only in the case of interfaces without tack-coat. Notwithstanding the results of Romanoschi and Metcalf [19], it is accepted that there is a dependency of interlayer bonding from normal stress, regardless if a tack-coat is applied or not at the interface. In fact, this dependency was expected from mechanical considerations on the influence of normal stress on frictional interface performance.

6.1.5.4 Specimen Conditions

Laboratory specimens and cores directly taken from the pavement have been compared by different authors [40, 46, 62] in terms of interlayer resistance. It has been shown that the average resistance from cores follows the same trend as from laboratory specimens, tested at the same temperature. However, the resistance of the cores was considerably lower than from the laboratory specimens.

As far as the curing time of the specimens is concerned, Canestrari et al. [40] and Rabiou and Morizur [73] showed that interlayer resistance increases with time, both with non-modified and modified emulsion tack-coat, for both laboratories made specimens and cores taken from the pavement.

Moreover, Raab and Partl [82] performed an in situ long-term performance investigation showing that interlayer shear properties are subjected to change over the life-time of a pavement as a function of, among other things, traffic and climatic history.

6.1.5.5 Further Evaluations

Raab [8] and Raab et al. [83] used Artificial Neural Network (ANN) as a modeling tool and demonstrated its capability of analyzing and predicting interlayer bond data, for determining influencing parameters and for establishing relationships between input and output parameters.

6.2 Inter-Laboratory Test on Interlayer Bonding of Asphalt Pavements

6.2.1 Introduction

As discussed in Sect. 6.1, interlayer shear resistance has recently attracted increasing interest worldwide due to the fact of steadily increasing requirements on pavement performance properties in terms of bearing capacity and durability as well as new innovative developments regarding pavement materials and construction. However, test methods available for assessing interlayer shear resistance and their reliability are under much discussion. In particular, interpretation of laboratory data in connection with data from real pavements are subject to uncertainties.

So far, no general consensus exists regarding test methods, assessment criteria and international standardization. Hence, RILEM TC 206-ATB TG 4 organized an interlaboratory test on interlayer bonding considering three different interface conditions of asphalt concrete pavements. The test focused on the comparison of shear test procedures, test conditions, specimen geometries and devices that allow determination of interlayer shear resistance in asphalt concrete.

The aim of this interlaboratory test was twofold:

- Determine the repeatability and reproducibility of interlayer shear tests proposed by the majority of the different participants.
- Determine correlations between different test procedures and evaluate the influence of different test conditions.

6.2.2 Trial Section

Samples were taken from a newly laid asphalt pavement near Ancona (Italy) in summer 2005, managed by the Università Politecnica delle Marche.

The trial section was 3.5 m wide and about 20 m long (Fig. 6.8) and composed of two layers. The lower layer had a thickness of 70 mm and the upper layer was 30 mm thick.

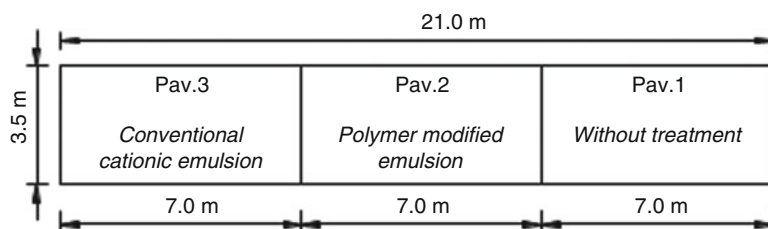


Fig. 6.8 RILEM trial pavement

Three different interface conditions were chosen.

- Pavement 1: without treatment (not hot on hot!)
- Pavement 2: pre-coated with a polymer modified emulsion
- Pavement 3: pre-coated with a conventional cationic emulsion.

6.2.2.1 Materials

The lower layer of the trial section consisted of an asphalt mixture type AC 16 binder 70/100 (EN 13108-1), whereas the upper layer was produced with an asphalt mixture type AC 11 surf 70/100 (EN 13108-1). The same aggregate type and the same bitumen were chosen to produce both mixes, but two different aggregate size gradations were used for the two layers. The mixture characteristics are shown in Table 6.1. Typical parameters of the tack-coat emulsions are presented in Table 6.2.

6.2.2.2 Trial Section Construction

The trial section construction started in July 2005.

In the first step, the existing unbound material was compacted to provide a regular and flat base surface for placing the two uppermost courses. Two different roller compactors with different dimensions and weights were used in succession to achieve a good level of compaction.

The same day, the binder course was laid and compacted (Fig. 6.9a), obtaining a layer thickness of about 70 mm.

Two weeks later, the polymer modified emulsion and the conventional cationic emulsion were manually applied with a brush (Fig. 6.9b) on the binder course of Pavement 2 and Pavement 3, respectively. The application rate was about 150 g/m² of residual bitumen for both Pavements.

After 6 h curing time for emulsions, the wearing course was laid (Fig. 6.9c) and compacted, obtaining a layer thickness of about 30 mm.

In Fig. 6.9d, the completed trial section is shown.

Table 6.1 Asphalt mixture, mixture design and laboratory tests result

Mixture type	AC 16 binder 70/100	AC 11 surf 70/100
Aggregate type	limestone	basalt
Binder content (% by weight)	5.2	4.6
Marshall specimen (50 blows):		
Air void (% V)	4.0	9.9
Stability (kN)	7.4	13.1
Flow value (mm)	3.2	3.2

Table 6.2 Bitumen emulsion characteristics

Emulsion type	Conventional cationic emulsion	SBS polymer modified emulsion
Residual binder content [%]	55	69
<i>Residue properties</i>		
Penetration @ 25 °C [dmm]	80	60
Viscosity @ 160 °C [Pa·s ⁻¹]	0.1	0.4
Softening point [°C]	70	40



Fig. 6.9 Phases of trial section construction



Fig. 6.10 Trial section after taking 100 and 150 mm cores from Pavement 1

Table 6.3 Time table of the construction and the sampling of the trial section

Work	Date
Construction of the trial section	July 2005
Drilling cores of Pavement 1	July 2005
Drilling cores of Pavement 2	February 2006
Drilling cores of Pavement 3	September 2006
Drilling cores of Pavement 1, 2 and 3 for additional tests	April 2007

6.2.2.3 Sampling

Asphalt cores were taken following a well-defined coring pattern with a clear documentation of the position of the cores (Fig. 6.10). Each core was marked by a special code defining its location in the trial section. The cores had a nominal diameter of 100 or 150 mm. The direction of laying was marked and identical to the direction of testing. The specimens were drilled at different times for Pavement 1, 2 and 3, as shown in Table 6.3.

6.2.3 Participating Laboratories

Fourteen laboratories participating on a voluntary basis are listed as follows:

- Amt der Kärntner Landesregierung – Bautechnik, Klagenfurt, Austria

- EMPA Dübendorf, Switzerland
- Institut für Straßenbau und Straßenerhaltung der TU Wien, Vienna, Austria
- Institut für Straßenwesen der TU Braunschweig, Braunschweig, Germany
- Laboratoire GEMH, Centre Universitaire de Génie Civil, Université de Limoges, Egletons, France
- National Center for Asphalt Technology, Auburn, AL, U.S.A
- National Cheng Kung University, Department of Civil Engineering, Tainan, Taiwan
- NCPE University of Nottingham, Nottingham, UK
- Nynas AB, Nynäshamn, Sweden
- Nynas UK AB, South Wirral, UK
- Road and Bridge Research Institute, Jagiellonska, Poland
- Transport Research Centre of CEDEX, Madrid, Spain
- Università Politecnica delle Marche, Ancona, Italy
- University of California Pavement Research Centre, Richmond, CA, U.S.A.

6.2.4 Experimental Investigation

6.2.4.1 Test Devices

Shear Test

The shear tests were performed either in pure direct shear configuration (Fig. 6.11) or direct shear configuration with normal stress (Fig. 6.12).

In the pure direct shear test, a shear displacement at a constant rate of about 50 mm/min is applied across a pre-defined plane, and the resulting shear force and the applied displacement are monitored. The test provides the shear force-shear displacement curve at the investigated plane. The shear stress is easily computable according to Eq. (6.7).

$$\tau = \frac{4 \times F}{\pi d^2} \quad (6.7)$$

where:

τ = shear stress (MPa)

F = shear force (N)

d = diameter of the specimen (mm)

The testing frame is fitted into an ordinary Marshall machine and allows testing specimens with 100 or 150 mm diameter. The so-called Leutner test is incorporated into several national standards or regulations [41, 84–86].

The working scheme of the direct shear device with normal stress is shown in Fig. 6.12. It was developed by the Università Politecnica delle Marche in Ancona [54], starting from the well-known direct shear test used in soil mechanics, due to its

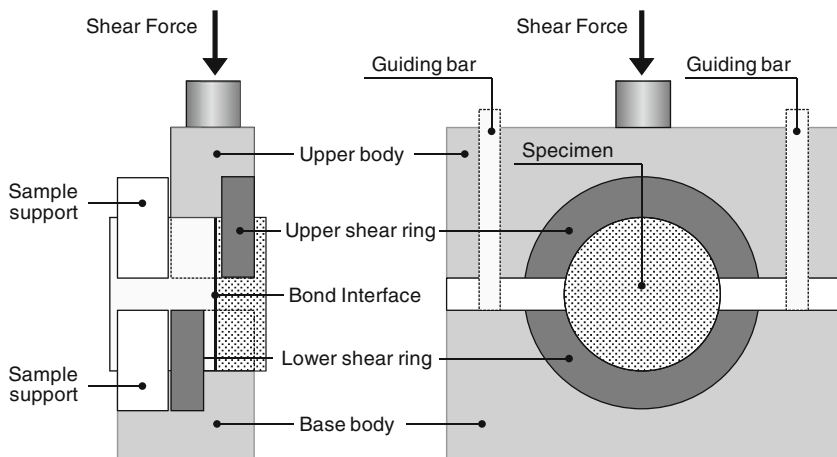


Fig. 6.11 Pure direct shear device (Leutner)

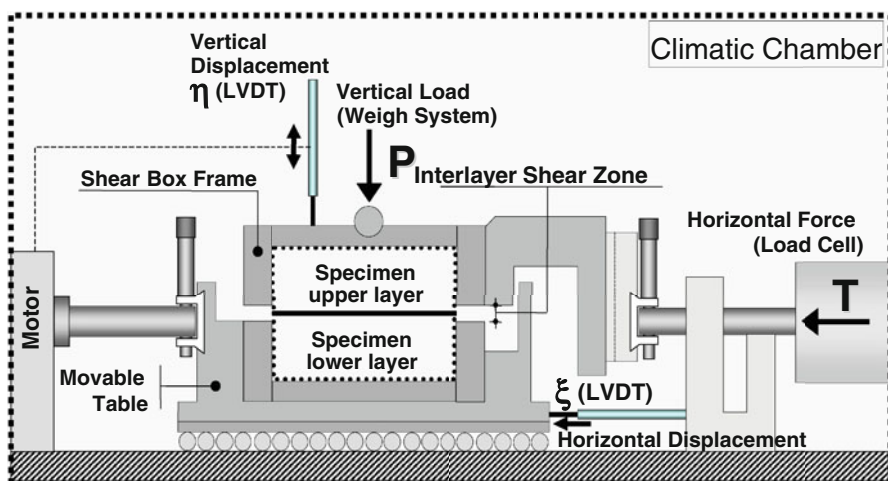


Fig. 6.12 Direct shear device with normal load (ASTRA)

extremely simple way of applying both normal and tangential loads on the specimen. The specimen is installed in two half-boxes separated by an unconfined interlayer shear zone in order to ensure that the shear force is applied in the weakest shear plane. A constant load, normal to the shear plane, is applied with a lever and weight system. A driving motor makes the movable table shift horizontally with a constant rate of 2.5 mm/min, while a load cell connected with the upper half-box reads and stores the shear force continually. At the same time, two transducers (LVDT) measure the shear (ξ) and normal (η) displacement of the specimen. The shear stress can be computed according to Eq. (6.8):

$$\tau = \frac{F}{A_{\text{eff}}} \quad (6.8)$$

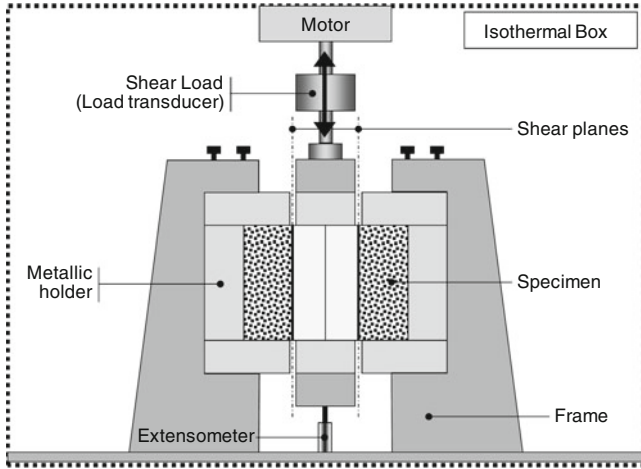


Fig. 6.13 Double Shear Test (DST)

where:

τ = shear stress (MPa)

F = shear force (N)

A_{eff} = effective cross sectional area at every instant (during the test, the interface contact area decreases because of the relative displacement of the two portions of the specimen).

During the test, dilatancy effects, which correlate the estimated volume changes to progressive shearing, may be evaluated referring to an additional dilatancy parameter d , defined as the ratio between normal and shear displacement increments $\Delta\eta$ and $\Delta\xi$, respectively, according to Eq. (6.9):

$$d = \frac{\Delta\eta}{\Delta\xi} \quad (6.9)$$

The whole apparatus is located in a climatic chamber with temperature and relative humidity control. The ASTRA device is specified in a national standard [58].

Double Shear Test

The Double Shear Test (DST) was developed by the University of Limoges [48, 49]. The working scheme of this device is shown in Fig. 6.13. It allows the investigation of asphalt concrete prismatic specimen behaviour in shear monotonic oligocyclic and fatigue loading, under controlled temperature conditions. The specimen ($70 \times 120 \times 50 \text{ mm}^3$) is composed of three parts (layers), two by two

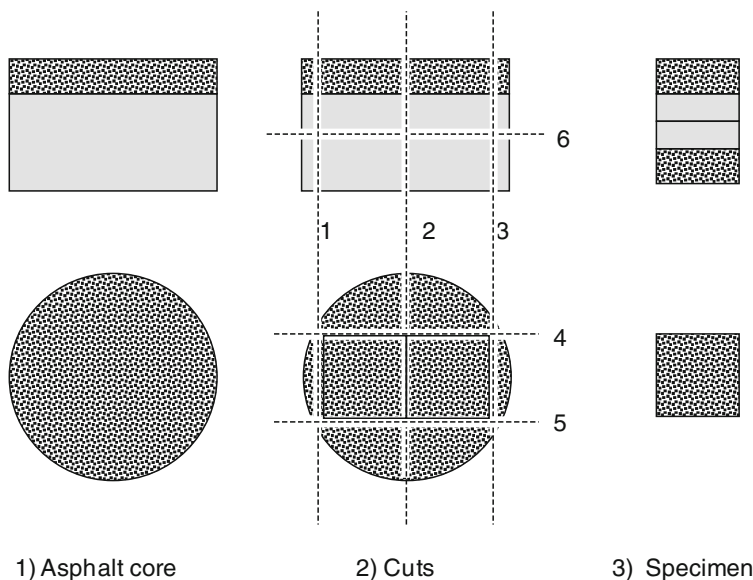


Fig. 6.14 Preparation of the test specimen for the DST

separated by interfaces. In the device working scheme the lateral upper and lower parts are fully fixed, whereas the central part is subjected to a displacement, which generates a shearing load at the interfaces. In particular, for fatigue tests, a symmetrical and sinusoidal alternate displacement is applied at the central layer, so that both interfaces are symmetrically subjected to a pure shearing load until failure is achieved. The shear relative displacement at the interfaces is measured by an extensometer. The advantage lies in the fact that both interfaces are symmetrically subjected to a quite pure shear load without extra bending moment. The specimen can be extracted from a core with a diameter of 150 mm by sawing two sub-specimens and gluing them to form the central part (Fig. 6.14).

The device can be used for unidirectional monotonic static shear test or for unidirectional cyclic fatigue shear test.

The shear stress is calculated with Eq. (6.10).

$$\tau = \frac{F}{2 \cdot a \cdot b} \quad (6.10)$$

where:

τ = shear stress (MPa)

F = shear force (N)

a, b = width and height of the specimen (mm)

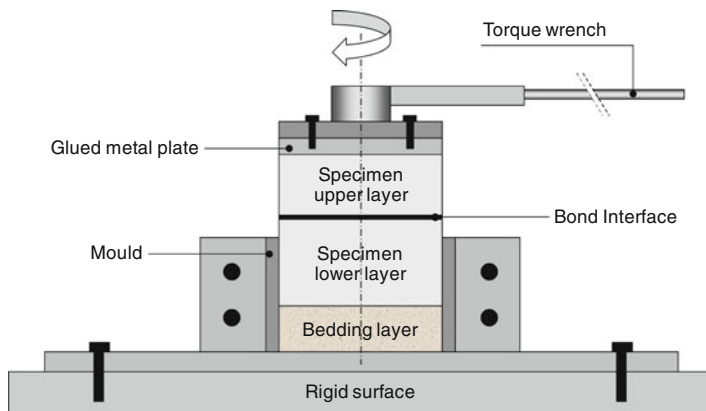


Fig. 6.15 Torque meter

Torque Bond Test

The working scheme of the torque bond test is shown in Fig. 6.15. The torque bond test can be carried out on site or in the laboratory and involves the measurement of the peak shearing torque, applied on cored specimens, to cause the de-bonding of a thin surface system from its substrate. For this purpose a steel plate is glued to the top surface of the specimen prior to the test. Then, a torque is applied manually to the top of the specimen by means of a torque wrench attached to the plate inducing a twisting failure mechanism in the bond layer. It is possible to test 100 or 150 mm diameter cores, but it is better to use the smallest specimens (100 mm) to limit the magnitude of the moment to break the bond. The value of the torque momentum at failure, M (in N·m), is recorded and the bond torque resistance τ (in MPa) can be calculated through the following equation:

$$\tau = \frac{12M \cdot 10^3}{\pi D^3} \quad (6.11)$$

where D is the core diameter (in mm). In the UK this procedure is put down in a draft of a national standard [87].

6.2.4.2 Test Program

The participating laboratories (see Sect. 6.2.3) carried out shear or torque tests on a total of about 1,400 cores. At each test condition seven specimens were tested. The test conditions varied in terms of diameter, test temperature, test speed, stress applied normal to the interface and age of the samples.

Table 6.4 Test Program; “x” in matrix denotes test conditions; (v: test speed, σ_n : normal stress)

Shear tests										
Diameter (mm)		150					100			
Temperature (°C)		10	20	25	30	40	10	20	30	40
σ_n (MPa) v:										
0	1.27 (mm/min)				x					
	12.7 (mm/min)				x					
	2.5 (mm/min)				x		x	x	x	
	25 (mm/min)				x					
	50 (mm/min)	x	x	x	x	x	x	x	x	
	200 (mm/min)	x	x		x					
0.049	2.5 (mm/min)			x	x	x				
0.2	2.5 (mm/min)						x	x	x	
0.483	2.5 (mm/min)								x	x
0.6	2.5 (mm/min)						x	x	x	
Torque bond tests										
0	180°/min	x	x		x		x	x	x	
	600 Nm/min						x	x	x	
Double Shear Test										
	10 Hz		x							

Table 6.5 Time table of testing

Phase	Pavement 1	Pavement 2	Pavement 3
original program	October 05–February 06	May 06–November 06	October 06–August 07
additional tests	May 07	May 07 November 07	November 07

The maximum shear load or torque momentum and the corresponding displacements were measured and the nominal maximum shear or torque shear stress were calculated. Additionally, the bulk density (EN 12697-6) of both layers was determined. The test program is given in Table 6.4.

As mentioned, the specimens were cored from the three pavement sections (Pavement 1, 2 and 3) at different times according to Table 6.1. In Table 6.5 the time table related to the interlayer bond testing is shown. It is divided into two different phases: the original program planned before the trial section construction and the additional tests planned at the end of the original program in order to study curing time effects.

6.2.5 Test Results and Data Analysis

The results of the interlaboratory testing program related to shear tests are summarized in Tables 6.6 and 6.7, whereas Tables 6.8 and 6.9 are, respectively, related to torque bond and Double Shear Test (DST).

Table 6.6 Shear tests (original program) – Number of laboratories (p), mean shear stress (τ) and mean displacement (Δu) at maximum shear load

			Pavement 1			Pavement 2			Pavement 3		
			p	τ	Δu	p	τ	Δu	p	τ	Δu
D	v	T	–	MPa	mm	–	MPa	mm	–	MPa	mm
mm	mm/min	°C	Shear test without normal stress								
150	50	10	3	1.07	1.50	3	2.04	2.56	3	2.22	2.35
		20	5	0.83	2.41	6	1.19	3.16	6	1.34	2.76
		25	2	0.61	2.55	2	0.88	4.30	2	1.02	4.31
		30	4	0.34	2.59	4	0.59	3.14	4	0.75	3.76
		40	1	0.19	2.26	1	0.34	3.47	1	0.36	3.58
100	50	10	3	1.09	1.19	3	2.34	1.54	3	2.43	1.63
		20	4	0.89	1.58	4	1.24	2.02	4	1.75	1.86
		30	3	0.51	2.16	3	0.74	2.51	3	0.95	2.55
100	2.5	10	4	0.61	1.56	4	1.15	2.31	4	1.24	2.28
		20	4	0.36	1.84	4	0.56	3.03	4	0.81	2.65
		30	4	0.23	2.63	4	0.34	2.76	4	0.44	2.98
150	2.5	30	1	0.28	4.11	–	–	–	–	–	–
150	1.27	30	1	0.26	4.51	–	–	–	–	–	–
150	25	30	–	–	–	–	–	–	1	0.67	4.66
150	12.7	30	–	–	–	–	–	–	1	0.56	5.78
Shear test with normal stress of 0.049 MPa											
150	2.5	25	1	0.50	2.00	1	0.47	2.71	1	0.50	3.19
		30	1	0.42	1.86	1	0.36	2.76	1	0.45	3.21
		40	1	0.25	1.55	1	0.25	2.93	1	0.26	2.25
Shear test with normal stress of 0.2 MPa											
100	2.5	10	1	0.65	2.14	1	0.95	3.11	1	1.35	4.66
		20	1	0.41	2.23	1	0.68	3.42	1	0.83	4.37
		30	1	0.32	2.86	1	0.37	2.96	1	0.50	3.97
Shear test with normal stress of 0.483 MPa											
100	2.5	30	1	0.61	5.49	–	–	–	–	–	–
		40	1	0.62	6.28	–	–	–	–	–	–

6.2.5.1 Bulk Density

The homogeneity of the test area is an important precondition for an interlaboratory study.

For this reason the bulk density of the specimen layers was chosen as a representative parameter and checked according to the European Standard EN 12697-6, (Procedure B: SSD) [88]. During the experimental activities of the participating laboratories the bulk density of the top (surface) and lower (binder) layer were determined after the shear or torque test. A critical analysis of the data of the top layer was necessary in order to identify and treat outliers or other irregularities (e.g. damage or deformation of the top layer after the shear test).

The statistical evaluation of the bulk density has been based on “Statistical analysis according ISO 5725-2 – Accuracy (trueness and precision) of

Table 6.7 Shear tests (additional program – curing time effects) – Number of laboratories (p), mean shear stress (τ) and mean displacement (Δu) at maximum shear load

			Pavement 1			Pavement 2			Pavement 3		
			p	τ	Δu	p	τ	Δu	p	τ	Δu
D	v	T	–	MPa	mm	–	MPa	mm	–	MPa	mm
mm	mm/min	°C	Shear test without normal stress								
150	200	10							1	2.48	2.54
		20							1	2.00	2.76
		30							1	1.18	3.81
150	50	10				1	2.49	2.17			
		20				1	1.59	2.69			
		30				1	0.98	2.99			
Shear test with normal stress of 0.6 MPa											
100	2.5	10	1	1.01	3.58	1	1.54	3.41	1	1.61	4.16
		20	1	0.73	5.13	1	1.00	5.95	1	1.05	4.80
		30	1	0.67	5.80	1	0.72	5.13	1	0.80	3.97
Shear test with normal stress of 0.2 MPa											
100	2.5	10				1	1.09	2.21			
		20				1	0.68	3.02			
		30				1	0.44	3.96			

Table 6.8 Torque bond test (original program) – Number of laboratories (p), mean shear stress (τ) and mean angle ($\Delta\alpha$) at maximum torque load

			Pavement 1			Pavement 2			Pavement 3		
D	v	T	p	τ	$\Delta\alpha$	p	τ	$\Delta\alpha$	p	τ	$\Delta\alpha$
mm	°/min	°C	–	MPa	deg	–	MPa	deg	–	MPa	deg
100	180	10	2	1.55 ^a		^b	^b	^b	^b	^b	^b
		20	2	1.01 ^a		2	1.68 ^a		2	1.82 ^a	
		30	2	0.61 ^a		2	0.73 ^a		2	0.88 ^a	
150	180	10	1	0.35		1	0.71		^b	^b	^b
		20	1	0.31		1	0.86		^b	^b	^b
		30	1	0.37		1	0.45		1	0.45	
		Nm/min									
100	600	10	1	1.01	3.81	1	1.51	4.96	1	1.70	5.79
		20	1	0.67	3.37	1	0.93	4.54	1	1.16	6.17
		30	1	0.34	3.34	1	0.71	4.70	1	0.62	5.90

^aThese mean values are not valid because the single values differ (outlier)

^bThe torque load exceeds the maximum capacity of the apparatus

Table 6.9 Double Shear Test (original program) – Number of laboratories (p), mean shear stress (τ) and mean displacement (Δu) at maximum shear load

			Pavement 3		
Size	v	T	p	τ	Δu
mm	mm/min	°C	–	MPa	mm
2 × 50 × 70	50	10	1	3.03	0.30
		20	1	2.42	0.31
		30	1	1.09	0.42

Table 6.10 Computed mean values (*m*), repeatability standard deviation *s_r* and reproducibility standard deviation *s_R* for the bulk density (g/cm³) of the two layers and number of samples (*n*) for both specimen diameters (*D*)

		Pavement 1		Pavement 2		Pavement 3	
Upper layer	<i>D</i>	100 mm	150 mm	100 mm	150 mm	100 mm	150 mm
	<i>n</i>	140	147	112	168	126	175
	<i>m</i>	2.208	2.199	2.220	2.207	2.217	2.215
	<i>s_r</i>	0.027	0.019	0.020	0.018	0.017	0.019
	<i>s_R</i>	0.029	0.026	0.037	0.019	0.018	0.021
Lower layer	<i>D</i>	100 mm	150 mm	100 mm	150 mm	100 mm	150 mm
	<i>n</i>	140	147	112	168	126	175
	<i>m</i>	2.250	2.258	2.273	2.258	2.260	2.255
	<i>s_r</i>	0.026	0.026	0.020	0.032	0.025	0.024
	<i>s_R</i>	0.034	0.028	0.022	0.031	0.026	0.028

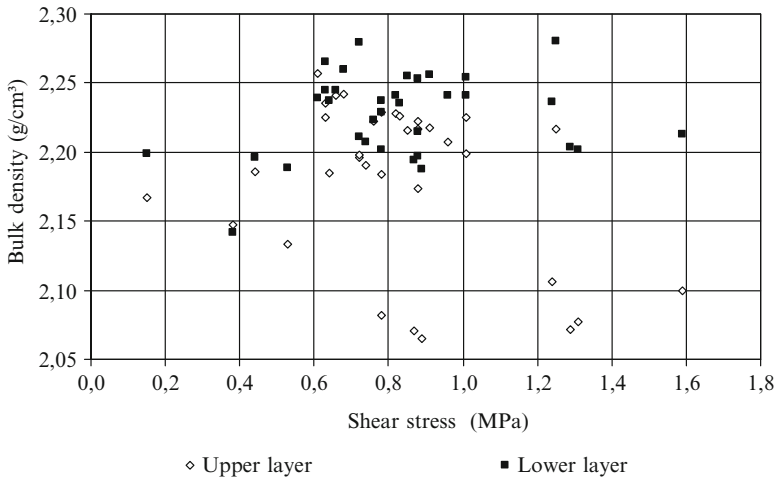


Fig. 6.16 Pavement 1, *D* = 150 mm, *T* = 20 °C, *v* = 50 mm/min, without normal stress, shear stress and bulk density of the *upper* and *lower* layer

measurement methods and results – Part 2: Basic method for the determination of repeatability and reproducibility of a standard measurement method” [89], and the results of the analysis are summarized in Table 6.10.

An example of the influence of the bulk density on the shear stress is given for Pavement 1 in Fig. 6.16, where it is evident that in this study the two parameters are not linked.

6.2.5.2 Precision of the Shear Test

The evaluation of the precision will give information about the quality of the data within each laboratory (repeatability standard deviation *s_r*) and among different

Table 6.11 Precision of maximum nominal shear stress “ τ ” (MPa)

Diameter	100 mm			150 mm		
	1	2	3	1	2	3
Pavement	1	2	3	1	2	3
Number of specimens	112	112	112	63	70	70
s_r (MPa)	0.25 τ	0.11 τ	0.09 τ	0.27 τ	0.14 τ	0.11 τ
s_R (MPa)	0.31 τ	0.22 τ	0.21 τ	0.34 τ	0.14 τ	0.17 τ

laboratories (reproducibility standard deviation s_R). The statistical analysis was done according to ISO5725-2 [89].

The results of the shear test were analysed if more than three laboratories had tested “identical” specimens under the same test conditions. In this part of the research program 539 specimens were tested without normal stress and evaluated.

- Diameter $D = 100$ mm, test speed $v = 2.5$ mm/min, temperatures $T = 10, 20, 30$ °C,
- Diameter $D = 100$ mm; test speed $v = 50$ mm/min, temperature $T = 20$ °C,
- Diameter $D = 150$ mm, test speed $v = 50$ mm/min, temperature $T = 20, 30$ °C,

The experimental results are summarized in Table 6.11. As can be seen, the values s_r and s_R are closely related to the mean values of the maximum nominal shear stress “ τ ” regardless of specimen diameter, test speed and temperature.

However, it must be considered that precision within one laboratory is based on testing the “same” sample by the same technician with the same device. Since shear tests are destructive tests, this condition was not fulfilled. Therefore the results of the shear test had to be checked with respect to their homogeneity within each pavement section.

For this purpose a new parameter, the shear stress factor f_τ^k is introduced. This parameter is calculated for each set of “ n ” specimens tested under the same conditions by a given laboratory. It is the ratio of the single maximum shear stress value τ_k of one specimen and the corresponding mean value of the considered set of specimens tested by the same laboratory, according to Eq. (6.12).

$$f_\tau^k = \frac{\tau_k}{\frac{1}{n} \sum_{i=1}^n \tau_i} \quad k = 1 \dots n \quad (6.12)$$

The value obtained for the shear stress factor has been associated with the location (spatially dispersed due to the random criterion of selection adopted for each set of specimens) in the pavement section of the corresponding specimen through a special code assigned to each core as explained in Sect. 6.2.2.3. For example, Fig. 6.17 shows the shear stress factor distribution in the pavement for one set of $n = 7$ randomly selected specimens that were tested by one given laboratory. In this way, repeating the same procedure for the overall set of specimens, the maximum shear stresses measured with different test conditions and/or equipment

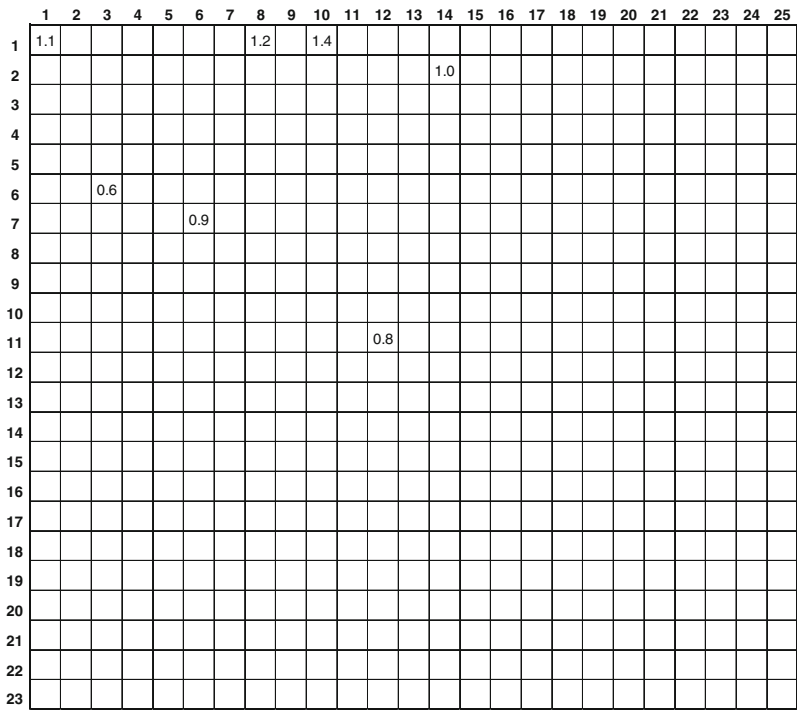


Fig. 6.17 Distribution of shear stress factors for one set of $n = 7$ specimens in the pavement

have been represented through a homogeneous parameter able to evaluate the specimen homogeneity within the pavement section.

In Fig. 6.18 the shear stress factor and the location in the pavement of each specimen are shown for Pavement 1 and $D = 150$ mm, as an example. The shear stress factor values describe the quality of interlayer bonding in the pavement section. For each position in the pavement pattern a different color has been chosen in order to better analyze the result distribution, according to the following criterion:

- black background when $f_{\tau}^k > 1.25$ (good interlayer bonding);
- grey background when $f_{\tau}^k = 0.75 \div 1.25$ (average interlayer bonding);
- light grey background when $f_{\tau}^k < 0.75$ (poor interlayer bonding).

In the further step of the analysis, a quadratic area of $3 \times 3 = 9$ cores (approximately corresponding to $0.34 \text{ m} \times 0.34 \text{ m} = 0.12 \text{ m}^2$ for cores with $D = 100$ mm and to $0.49 \text{ m} \times 0.49 \text{ m} = 0.24 \text{ m}^2$ for cores with $D = 150$ mm) has been considered in order to determine the homogeneity through the standard deviation σ of the shear stress factor for a minimum of seven cores.

Starting from the shear stress factors shown in Fig. 6.18, the corresponding standard deviation values are calculated as follows: for a given position (i, j) the standard deviation is based on the results reported in the nine positions $(i + p, j + p)$

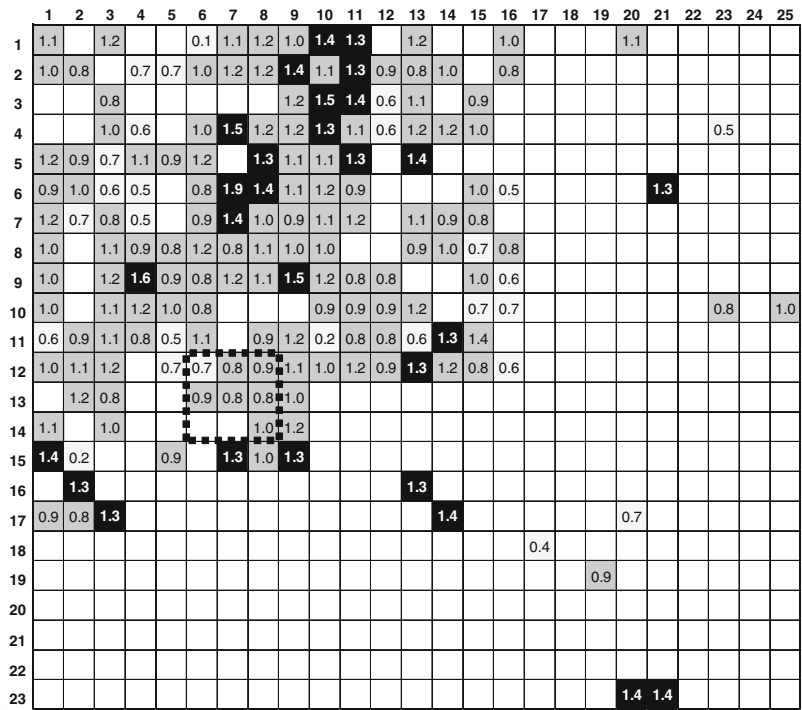


Fig. 6.18 Shear stress factors for Pavement 1 and cores with $d = 150$ mm. Encircled area corresponds to the highest degree of homogeneity (minimum standard deviation)

with “p” from 0 to 2. The calculated standard deviation values for cores of Pavement 1 with a diameter of 150 mm are represented in Fig. 6.19, where a grey background is used for those positions in which the standard deviation is significant, i.e., determined with at least seven values of the shear stress factor among the nine positions considered for each calculation. Only these values have been considered representative for further evaluations. In this sense, the minimum standard deviation ($\sigma = 0.11$) corresponding to the position $(i, j) = (6, 12)$ and obtained with the seven values indicated in Fig. 6.18 in correspondence of the area with the highest degree of homogeneity, is encircled black. In fact, for a given area of the pavement section, a good homogeneity is assumed when asphalt cores have shear stress factors that tend to the same value and the corresponding standard deviation σ converges to a minimum value.

The same procedure has been repeated for all pavements (1, 2, 3) and for both specimen diameters (100, 150 mm) giving the results shown in Table 6.12.

The minimum standard deviation of the shear stress factors has been chosen as a reference parameter in order to calculate the repeatability standard deviation s_r and the reproducibility standard deviation s_R of the maximum shear stresses in the case of absolutely homogeneous area. In detail, both series of six values for s_r and s_R

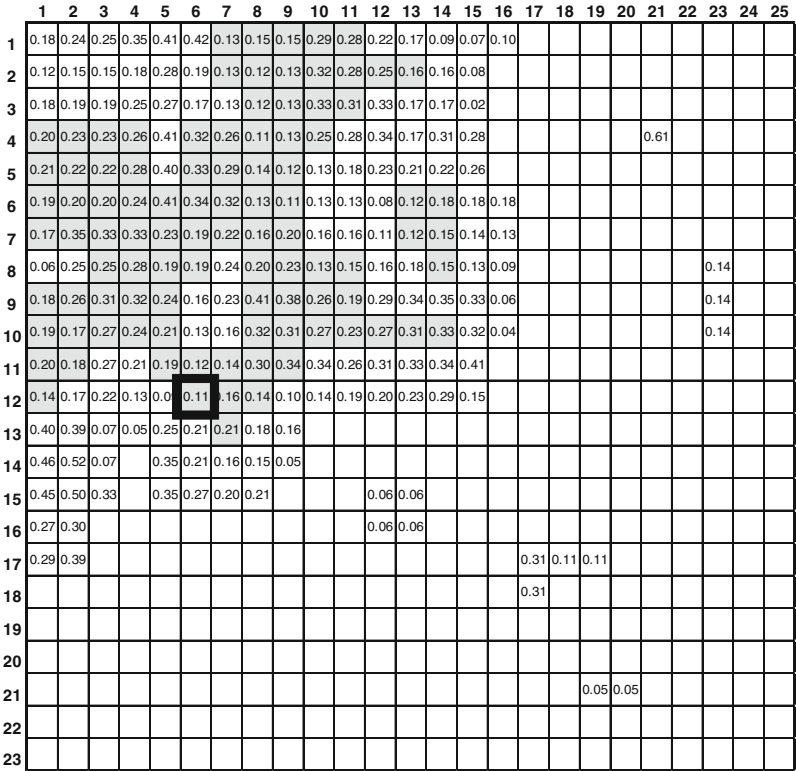


Fig. 6.19 Significant standard deviations (*grey background*) at position (i, j) determined from the shear stress factors at nine positions ([i, i + 2], [j, j + 2]) for Pavement 1 and cores with d = 150 mm. Encircled value is the minimum standard deviation $\sigma = 0.11$ for this case

Table 6.12 Minimum standard deviation for the shear stress factor

Pavement	Specimen diameter, mm	Minimum standard deviation for the shear stress factor
1	100	0.070
	150	0.107
2	100	0.028
	150	0.038
3	100	0.035
	150	0.024

(Table 6.11) have been linearly correlated with the corresponding minimum standard deviation for the shear stress factor summarized in Table 6.12.

The basic principle is to obtain the repeatability standard deviation s_r and the reproducibility standard deviation s_R not affected by the unhomogeneity of the pavement section. To this end the statistical parameters s_r and s_R for a perfectly homogeneous pavement are calculated through the regressed values obtained with a minimum standard deviation for the shear stress factor equal to 0. Such values

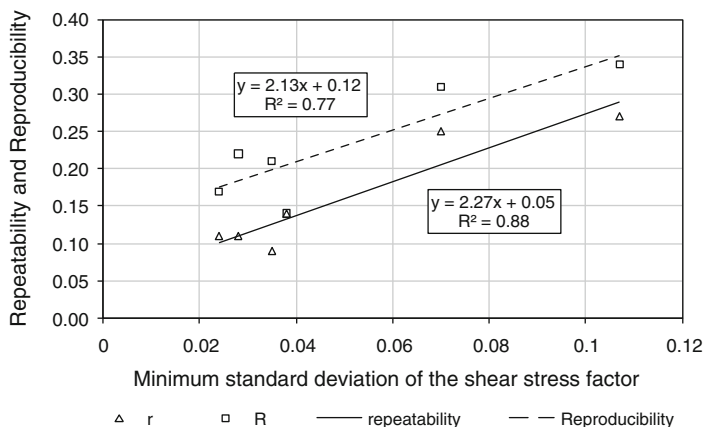


Fig. 6.20 Correlation between the repeatability standard deviation s_r and the reproducibility standard deviation s_R of the maximum shear stresses and the corresponding minimum values of the standard deviation for the shear stress factor

correspond to the intercept of the regression lines shown in Fig. 6.20. Finally it was found that the precision of the shear test in case of absolutely homogeneous area of the trial section is:

- Repeatability standard deviation: $s_r = 0.05 \tau$ (MPa)
- Reproducibility standard deviation: $s_R = 0.12 \tau$ (MPa).

6.2.5.3 Precision of the Torque Bond Test

The torque bond tests were performed by only two laboratories, considering three different conditions:

- Diameter $D = 100$ mm; test speed $v = 180^\circ/\text{min}$, temperature $T = 10, 20, 30^\circ\text{C}$;
- Diameter $D = 150$ mm; test speed $v = 180^\circ/\text{min}$, temperature $T = 10, 20, 30^\circ\text{C}$;
- Diameter $D = 100$ mm, test speed $v = 600 \text{ Nm}/\text{min}$, temperatures $T = 10, 20, 30^\circ\text{C}$.

A total of 84 specimens were tested and only the first condition ($D = 100$ mm, $v = 180^\circ/\text{min}$, $T = 10, 20, 30^\circ\text{C}$) was investigated by both laboratories. According to the criterion established for the precision investigation of the shear test results (at least three laboratories should test “identical” specimen under the same test condition), it was impossible to perform the statistical analysis of the torque test results.

The impracticality to obtain repeatable and reproducible values in this case is also due to three important limitations of this kind of test:



Fig. 6.21 Torque bond test: failures inside the *upper layer*

- The failure occurs in correspondence of the weakest plane of the specimen, and this means that it does not necessarily occur at the interface, as shown in Fig. 6.21. Thus, in these cases, it is not possible to record the results because they do not represent the bond torque resistance.
- The results obtained by the two laboratories at the same conditions ($D = 100$ mm, $v = 180^\circ/\text{min}$, $T = 10, 20, 30^\circ\text{C}$) remarkably differ from the line of equality, and higher torque resistance values were obtained by lab 1 in comparison with lab 2 (Fig. 6.22).
- In some cases, it was impossible to perform the test because the torque load exceeded the maximum apparatus capacity, as indicated in Table 6.8.

6.2.6 Interpretation and Discussion

6.2.6.1 Influence of Test Parameters on Shear Test Results

Influence of Specimen Diameter

For the determination of the influence of specimen diameter, nine pairs of values were available. The mean shear stress values for all three pavements tested at a speed of 50 mm/min and temperatures of 10, 20 and 30 °C were evaluated based on 616 cores. The evaluation showed that all shear stress values for all temperatures tested with cores of 150 mm diameter were lower than the results of the test carried out with 100 mm asphalt cores. The correlation is given by Eq. (6.13), and the corresponding coefficients are listed in Table 6.13.

$$\tau_{D=100\text{ mm}} = a \cdot \tau_{D=150\text{ mm}} \quad (6.13)$$

Fig. 6.22 Comparison between lab 1 and lab 2 in terms of torque resistance

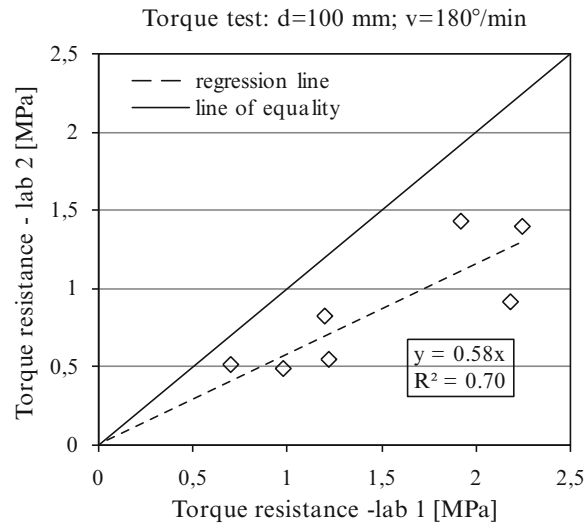


Table 6.13 Coefficient a and R² of the different pavements

Pavement	a	R ²
1	1.07	0.86
2	1.13	0.99
3	1.16	0.94

It appears that the factor “a” depends on the interface condition. Moreover, as expected, the presence of emulsion mitigates the scatter of shear stress at the interface giving higher R² values for Pavements 2 and 3. The linear regression for all samples has a coefficient of R² = 0.970. It was found that shear stress of 100 mm cores is about 14% higher than shear stress of 150 mm cores.

Influence of the Test Speed

A total of 77 cores were tested. The test speed used by the different laboratories for the shear tests ranged from 1.27 to 200 mm/min. In Fig. 6.23, the shear stress results obtained for Pavement 3 considering five different test speeds are presented.

Figure 6.23 indicates in the investigated speed range a strong power function type of relationship with a high regression coefficient (R² = 0.99).

The other data were evaluated in a similar way and lead to the general equation for all temperatures and pavements (D = 150 mm).

$$\tau = 0.41 \cdot v^{0.22} \tag{6.14}$$

Again, the regression coefficient (R² = 0.99) is very high. Note that the exponent of the power function slightly changed as compared to Fig. 6.23. Hence, if one

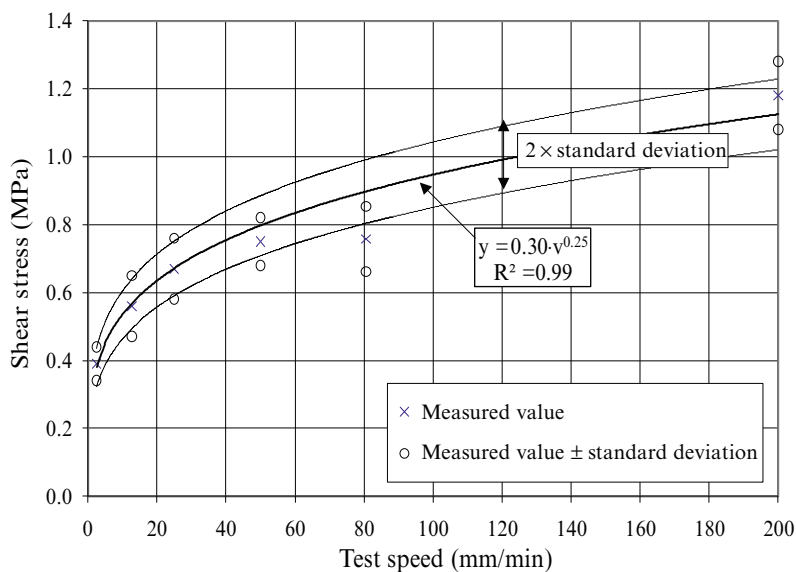


Fig. 6.23 Correlation between test speeds and shear stress at 30 °C and D = 150 mm for Pavement 3

shear stress value and its test speed are known, shear stress can be calculated by Eq. (6.15) for whatever test speed as follows.

$$\tau_{vx} = \tau_{v1} \cdot \left(\frac{v_x}{v_1} \right)^{0.22} \quad (6.15)$$

where:

τ_{v1} = shear stress at test speed 1

v_1 = test speed 1

τ_{vx} = shear stress at test speed x

v_x = test speed x

The Influence of Normal Stress

In order to study the influence of nominal normal stress on the maximum shear stress, the results from one laboratory were considered. The results obtained for the three pavements and for three temperatures (10, 20 and 30 °C) were analysed, for a total of 63 specimens.

Figure 6.24 allows the determination of two important contributions to the maximum shear stress: the cohesion and the friction. The cohesion is the value of τ_{max} when the normal load is not applied. Therefore the cohesion is represented by

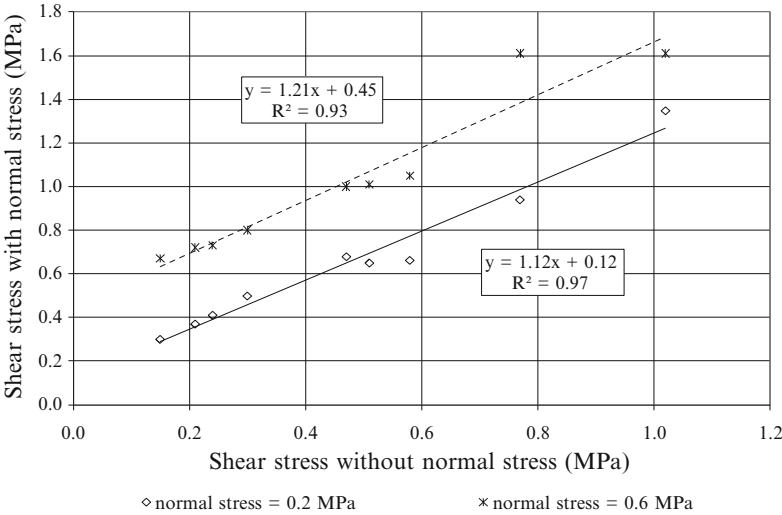


Fig. 6.24 Correlation between shear stress with and without normal stress applied

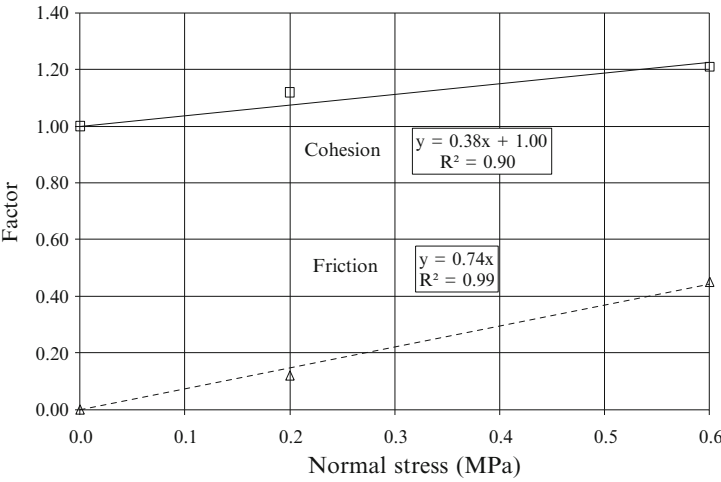


Fig. 6.25 Cohesion and friction contribution as a function of the regression parameters calculated for the correlation between shear stress with and without normal stress applied

the x axis, whereas the friction is given by the intercept of the regression lines in Fig. 6.24 obtained with two values of normal stress (0.2 and 0.6 MPa).

Further evaluations of the parameters of the regression lines (Fig. 6.25) show that these parameters were linearly dependent on the normal stress and lead to the Eq. (6.16) as functions of a general value of σ_n .

$$\tau_{\sigma n} = (1 + 0.38 \cdot \sigma_n) \cdot \tau_{\sigma 0} + (0.74 \cdot \sigma_n) \tag{6.16}$$

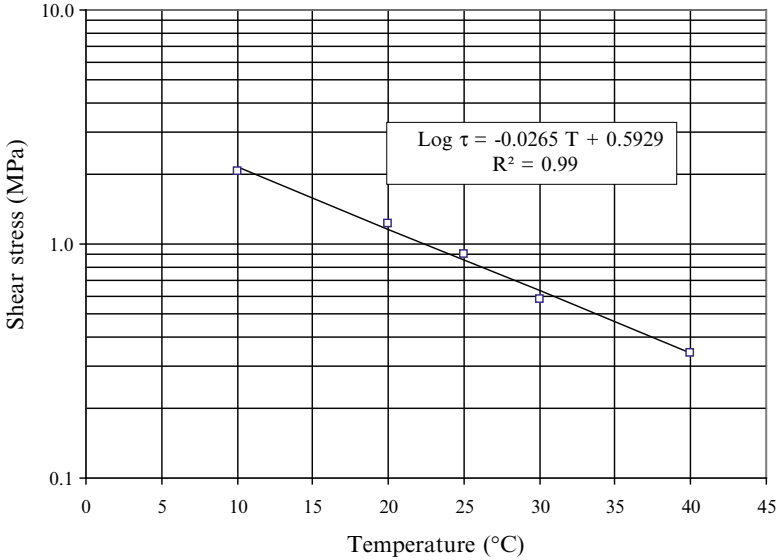


Fig. 6.26 Correlation between shear stress without normal stress and temperatures for Pavement 2 D = 150 mm, v = 50 mm/min

where:

- $\tau_{\sigma n}$ shear stress with normal stress σ_n (MPa)
- $\tau_{\sigma 0}$ shear stress without normal stress (MPa)
- σ_n normal stress (MPa)

Influence of the Temperature

In this investigation, all laboratories carried out the tests at various temperatures. One laboratory used five temperatures (10, 20, 25, 30 and 40 °C). As an example, the results of Pavement 2 are shown for D = 150 mm and v = 50 mm/min in Fig. 6.26.

A relation between shear stress and temperatures was found as follows:

$$\tau_{T_x} = 10^{a \cdot T_x + b} \tag{6.17}$$

Table 6.14 presents the corresponding regression coefficients. From these tests it appears that all pavements follow the same temperature dependency. The mean value of coefficient “a” is −0.023 (±0.004). Figure 6.27 is an example for D = 100 mm and v = 50 mm/min and was produced by using these coefficients.

Table 6.14 Coefficient a and b for all pavements according to Eq. (6.17)

D	v	Pavement 1			Pavement 2			Pavement 3		
		a	b	R ²	a	b	R ²	a	b	R ²
mm	mm/min	Shear test without normal stress								
100	2.5	−0.021	−0.013	1.00	−0.026	0.308	0.99	−0.023	0.334	0.99
100	50	−0.016	0.211	0.92	−0.025	0.612	1.00	−0.020	0.611	0.97
150	50	−0.026	0.369	0.94	−0.026	0.586	1.00	−0.026	0.638	0.99

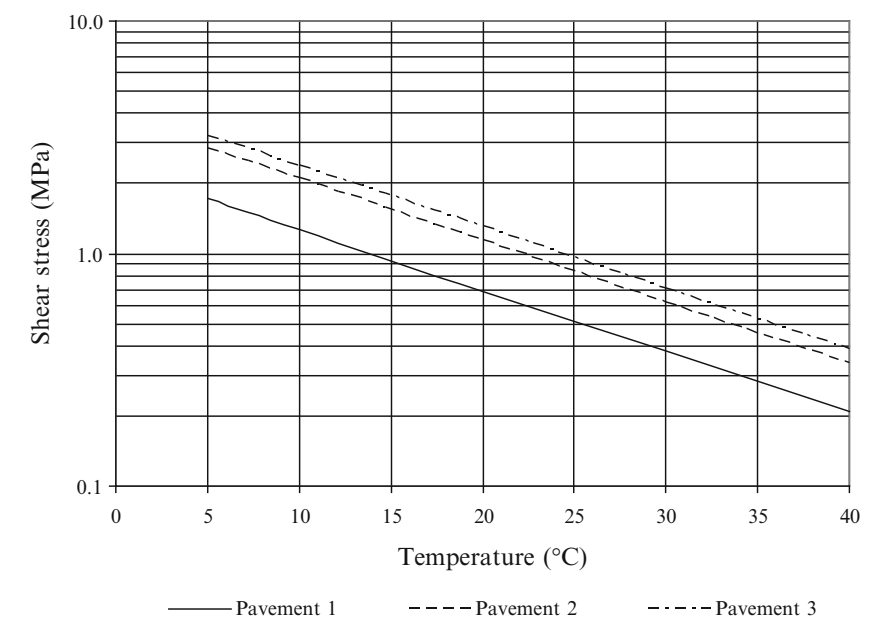


Fig. 6.27 Shear stress as a function of temperature ($D = 150\text{ mm}$, $v = 50\text{ mm/min}$)

Influence of the Sample Age

The influence of the sample age in the absence of traffic was checked for Pavement 2. Tests were carried out by two laboratories. The results are shown in Table 6.15 and Fig. 6.28.

The evaluations showed an increase of the shear stress. The fact that both laboratories found an increase of shear stress (up to 27%) shows the influence of sample age. However the amount of increase was different between the two laboratories, which is partly due to the fact that the testing conditions were not identical.

Table 6.15 Influence of the sample age on shear stress

Laboratory	1		2	
Test conditions	D = 150 mm, v = 50 mm/min, normal stress = 0 MPa		D = 100 mm, v = 2.5 mm/min, normal stress = 0.2 MPa	
Test date	May 2006	November 2007	June 2006	November 2007
Temperature	Shear stress (MPa)			
10 °C	2.03	2.49	0.95	1.09
20 °C	1.23	1.59	0.68	0.68
30 °C	0.58	0.98	0.37	0.44

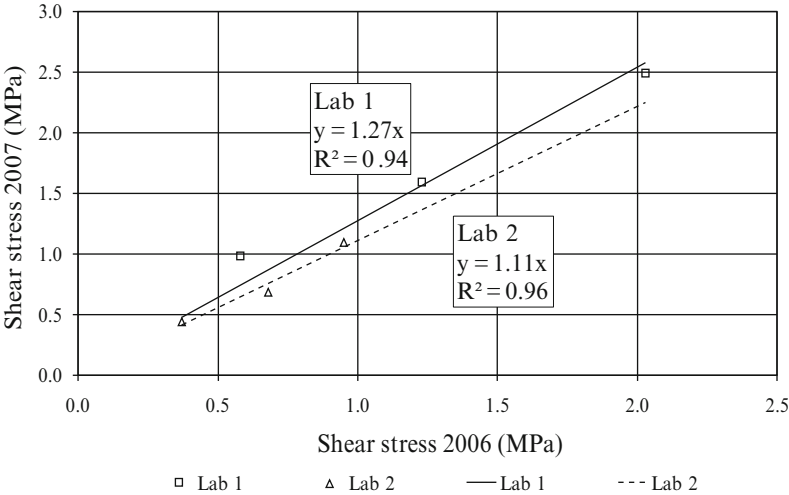


Fig. 6.28 Correlation of shear stress measured at different age

6.2.6.2 Influence of Test Parameters on Torque Bond Test Results

Influence of the Temperature

The influence of the temperature on the bond torque test could be identified following the same procedure carried out for the shear test through the determination of the regression parameters (a and b) introduced in Eq. (6.17).

However, in this case, the limitations of the torque test described in Sect. 6.2.5.3 do not allow a reliable determination of the relationship between temperature and torque resistance for all the three conditions investigated. In fact, the sole condition that showed acceptable results and thus tolerable regression parameters is the condition with D = 100 mm, v = 600 Nm/min and T = 10, 20, 30 °C. The temperature dependency of the torque bond test for this condition is shown in Fig. 6.29, and the corresponding regression parameters are listed in Table 6.16.

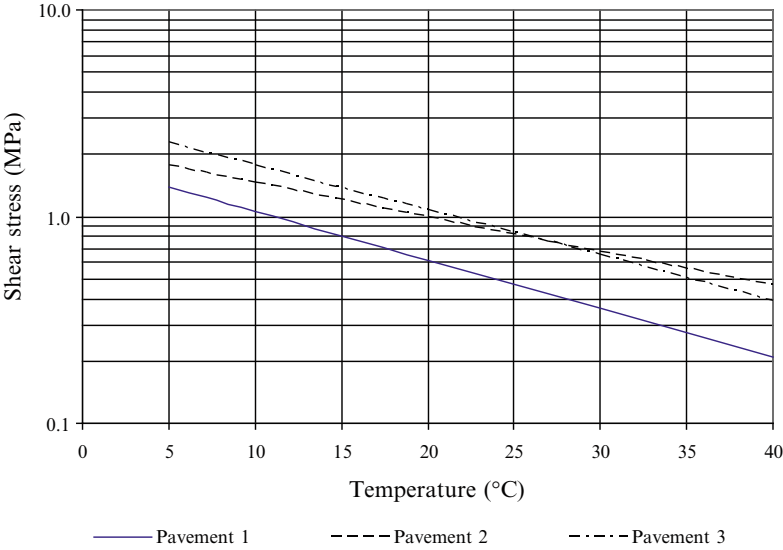


Fig. 6.29 Torque bond test – temperature dependency

Table 6.16 Torque bond test – parameters “a” and “b” for all pavements according to Eq. (6.17)

		Pavement 1			Pavement 2			Pavement 3		
D, mm	V, Nm/min	a	b	R ²	a	b	R ²	a	b	R ²
100	600	−0.024	0.260	0.98	−0.016	0.327	0.97	−0.022	0.467	0.98

Comparison Between Torque Bond Test and Shear Test

A reliable comparison between torque bond test and shear test results does not seem reasonable due to the large difference between the results obtained by the two laboratories that performed bond torque tests (see Sect. 6.2.5.3).

An attempt to find a relationship between torque and shear results (Table 6.17) is shown in Fig. 6.30 which shows that the two laboratories that performed torque tests provided different relationships with the shear results making it impossible to find a unique correlation between the two test methods. Moreover, the limited and not statistically representative number of data provided very low coefficients of determination (R^2) for both curves shown in Fig. 6.30.

6.2.6.3 Influence of Test Parameters on Double Shear Test Results

The static application of the Double Shear Test (DST) was performed by a single laboratory and with the specimens cored from only Pavement 3. Therefore, the temperature dependency and the relationship with the shear test can be obtained only for this pavement.

Table 6.17 Mean shear stresses (static shear test) and torque bond test resistances

Type of test		Shear test	Torque bond test	
Laboratory			1	2
Test conditions		D = 100 mm v = 50 mm/min	v = 180°/min	
	T °C	Without normal stress		
Pavement 1	10	1.09	2.18	0.91
	20	0.89	1.20	0.82
	30	0.51	0.70	0.51
Pavement 2	10	2.34	>2.42	1.30
	20	1.24	1.92	1.43
	30	0.74	0.98	0.49
Pavement 3	10	2.43		
	20	1.75	2.25	1.40
	30	0.95	1.22	0.54

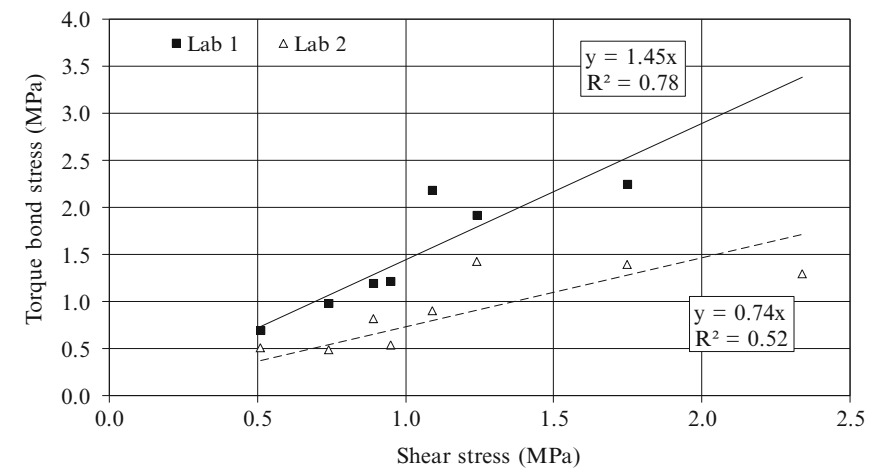


Fig. 6.30 Comparison between shear test (v = 50 mm/min, without normal stress) and torque bond test (v = 180°/min), d = 100 mm, T = 10, 20, 30 °C

Influence of the Temperature

Also in this case, the influence of the temperature on the Double Shear Test (static application) is identified following the same procedure as carried out for the shear test. The temperature dependency obtained for Pavement 3 is shown in Fig. 6.31 and the corresponding regression parameters, introduced with Eq. (6.16), are shown in Table 6.18. The value of parameter “a” is close to the mean value of shear tests without normal stress obtained for Pavement 3 (Table 6.14).

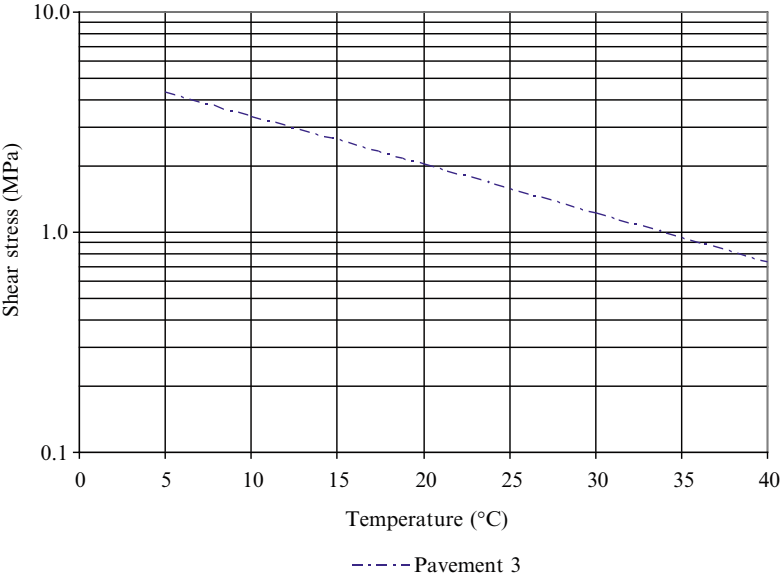


Fig. 6.31 Double shear test – temperature dependency

Table 6.18 Double shear test – parameters “a” and “b” for Pavement 3 according to Eq. (6.17)

Sample size	v	Pavement 3		
		a	b	R ²
mm	mm/min	Double Shear Test		
2 × 50 × 70	50	−0.022	0.745	0.91

Table 6.19 Shear stress and shear area for Pavement 3 (test speed: 50 mm/min)

Type of test	Shear test		Double Shear Test
Number of laboratories	3–6	3–4	1
Sample size	D = 150 mm	D = 100 mm	2 × 50 × 70 mm
Shear area	16,300 mm ²	7,100 mm ²	2 × 3,500 mm ²
Test temperature	Mean shear stress (MPa)		
10 °C	2.22	2.43	3.03
20 °C	1.34	1.75	2.42
30 °C	0.75	0.95	1.09

Comparison Between Double Shear Test (Static Application) and Shear Test

The test results, for Pavement 3, obtained with different test devices and/or specimen sizes are listed in Table 6.19 and shown in Figs. 6.32 and 6.33.

Table 6.19 and Fig. 6.32 show that the Double Shear Test (DST) provides higher shear stress values than the shear test. When reporting all these values as a function

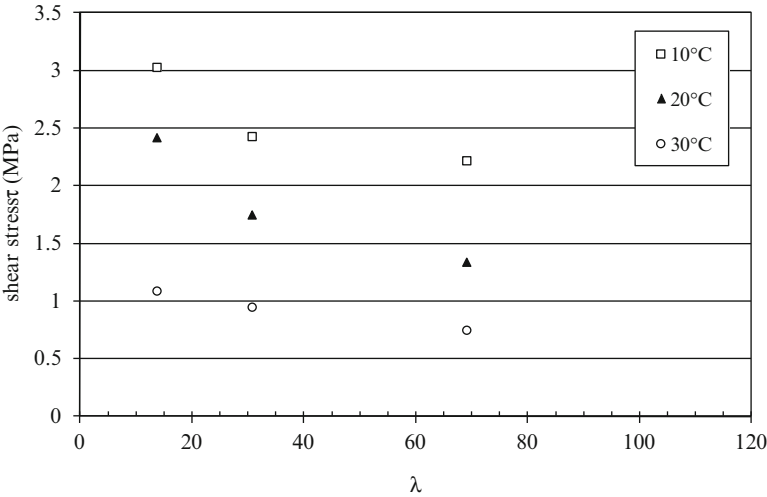


Fig. 6.32 Pavement 3 – comparison between shear test and DST ($v = 50$ mm/min, without normal stress)

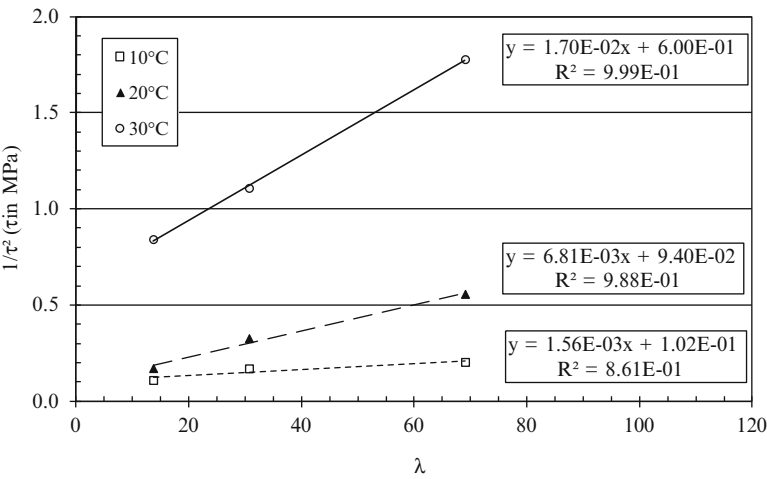


Fig. 6.33 Pavement 3 – Size/scale effect for shear test and DST ($v = 50$ mm/min, without normal stress)

of a size ratio λ defined as S (shear area) vs. $D_{\max}^2$ (square maximal nominal size equal to 16 mm), a clear size/scale effect can be observed according to quasi-brittle material behaviour used for 1D flexible beam [90], represented by the following equation:

$$\tau_{SR} = \frac{\tau}{\sqrt{1 + k \cdot \lambda}} \tag{6.18}$$

Table 6.20 Fatigue test for Pavement 3 – Imposed peak relative displacement (Δu), failure at 50% (Nf), initial dissipated energy per cycle (DE)

Size	Frequency	T	p	Δu	NF	DE
mm	Hz	°C	–	μm		J/mm^2
$2 \times 50 \times 70$	10	20	1	4	2,501,607	853 E-6
			1	7.5	171,380	345 E-5
			1	10	19,371	593 E-5
			1	15	2,931	135 E-4
			1	20	1,657	205 E-4

where:

- τ_{SR} is the measured size-reduced strength of the interface;
- τ is the strength of the interface;
- k is a constant when geometrically similar test conditions are considered;
- λ is a non-dimensional size parameter equal to S/D_{max}^2 .

Starting from Eq. (6.18), for a given strength of the interface τ , the following linear correlation between $1/(\tau_{\text{SR}})^2$ and λ can be obtained:

$$\frac{1}{(\tau_{\text{SR}})^2} = \frac{1}{\tau^2} (1 + k \cdot \lambda)$$

(6.19)

Good correlations between the three sample sizes are obtained in Fig. 6.33 according to Eq. (6.19). The good linearity obtained in the graph enables to explain the variation of shear stress levels measured with different test methods as a function of the size parameter λ . In conclusion, the results obtained in this study show a clear influence of the size effect. However, due to the limited data available, a specific experimental investigation should be planned in order to identify both the parameters k and τ introduced in Eq. (6.18).

6.2.6.4 Fatigue Test Performed with the Double Shear Test

In order to introduce further prospects for the study of the interlayer shear behaviour of double-layer specimens with dynamic devices, the fatigue properties of the Pavement 3 specimens have also been investigated with the Double Shear Test.

Loading level in controlled displacement mode, initial dissipated energy level and some results in terms of conventional fatigue lifetime (Nf), are summarized in Table 6.20 and Fig. 6.34.

The fatigue behaviour can be represented, as usual, in a log-log scale, e.g., as loading or initial dissipated energy per cycle vs. conventional lifetime (Nf).

A good correlation can be observed, from oligocyclic to high cycle fatigue, for both approaches.

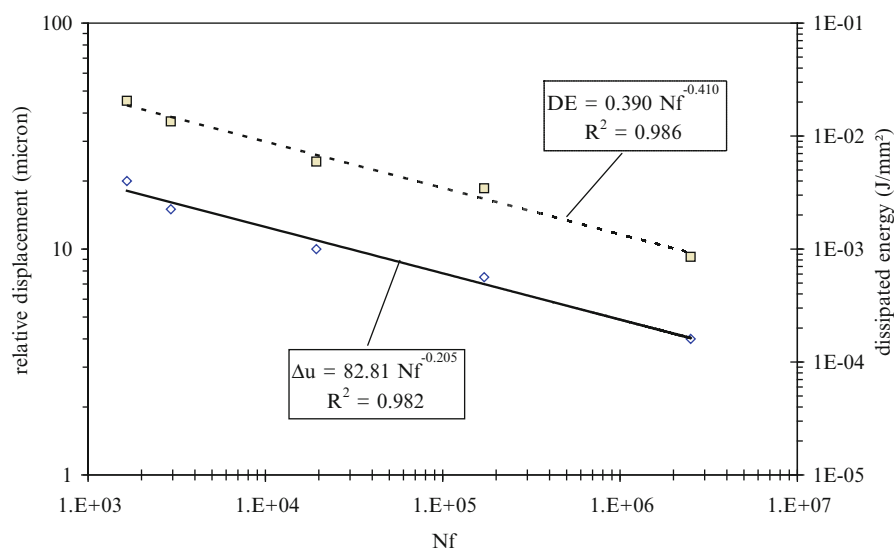


Fig. 6.34 Fatigue laws: peak of relative displacement loading and dissipated energy vs. failure criteria at 50% decreasing stiffness

6.3 Conclusions

Interlayer bonding at the interfaces of multi-layered bituminous systems has been recognized as a key issue for the evaluation of stress-strain distribution in road pavements produced by traffic loads that are responsible for steadily increasing requirements on pavement performance properties.

For this reason, correct assessment of interlayer bonding is of primary importance, calling for further research efforts in order to improve the lack of correlation and/or harmonization among test methods.

In this context, the aim of the RILEM interlaboratory test program was to compare test methods for the assessment of interlayer bonding of asphalt pavements. The conclusions from this investigation can be summarized as follows.

- Generally good agreement exists between different unidirectional monotonic static shear tests.
- The achieved precision of the different shear tests is satisfactory; it was found that the precision of the shear test in an absolutely homogeneous area of the trial section is:
 - Repeatability standard deviation: $s_r = 0.05 \tau$ (MPa)
 - Reproducibility standard deviation: $s_R = 0.12 \tau$ (MPa)
- The influence of the temperature, the size of the specimen and the test speed could be mathematically evaluated and clear correlations could be found.

- As for the influence of the specimen diameter, it was found that the maximum shear stress values for all temperatures tested with cores of 150 mm diameter were lower than the test results obtained with 100 mm asphalt cores.
- The presence of emulsion seems to mitigate the scatter of maximum shear stress at the interface.
- A relationship was found that allows estimating the influence of the test speed on maximum shear stress.
- Friction parameters appeared to be approximately linearly dependent on normal stress. This was also true for the slope of the linear regression between maximum shear stress with and without normal stress.
- Regardless of the interface conditions, the maximum shear stress appears to follow the same temperature dependency.
- Within the 18 months from first to second sampling, an increase of the maximum shear stress with sample age was observed.
- No statistically significant correlation between torque and shear test was obtained, also due to the dispersion and the limited number of torque test results achieved by the participating laboratories.
- A linear relation between Double Shear Test (DST) and shear test was found for the temperature range between 10 and 30 °C, with generally about 30% higher values for DST than for shear test.
- The results obtained in this study show a clear influence of the size effect. However, due to the limited data available, a specific experimental investigation should be planned in order to identify proper values for scale effect parameters.

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Chapter 7

Hot Recycling of Bituminous Mixtures

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Abstract This chapter first presents the results of a survey performed on the practices used in Europe. This survey is considered a first report on the subject and could be usefully completed by the states of the art prepared in the framework of the Re-road and Direct-Mat European projects. Then the chapter addresses the key issue of Reclaimed Asphalt (RA) manufacture in laboratory considered within

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the group as an important point to assess recyclability of bituminous mixtures. To answer this question, common research has been launched on the basis of an interlaboratory test programme to set a relevant laboratory ageing of bituminous mixtures protocol. The choices of the tested protocol, the setting of the interlaboratory test programme as well as the results obtained, are presented in this chapter. A conclusion in the form of a recommendation for laboratory ageing of bituminous mixtures is given at the end of the chapter.

Keywords Bituminous mixtures • Ageing • Test protocol • Reclaimed asphalt • Recycling • Bitumen • Round robin test • Infrared spectroscopy

7.1 Introduction

With more and more difficult access to easy oil, shortages of good quality aggregates, and the general conscience towards environmental issues, recycling of asphalt pavement is gaining more and more attention, particularly in Europe. Hence, European Union encourages road authorities to launch huge projects to assess some of the issues involved in recycling materials in asphalt pavements.

Thus, the SAMARIS project (Sustainable and Advanced Materials for Roads and Infra-Structures, Piau and Christensen [20]), from December 2002 to March 2006, was initiated by FEHRL, the Forum of Europe's National Road Research Centres. It included two areas, one dealing with pavements and the other dealing with highway structures. One of the key objectives was to work on a safe use of alternative materials to encourage the use of recycled materials in pavements, by detailing how such materials could be selected and tested in order to secure satisfactory performance, both environmentally and functionally.

The more technology oriented project SCORE (Superior COLD REcycling), involved eight partners for 3 years (2002–2005) to focus on the basic understanding of cold processes, using micro-emulsions and foams to recycle asphalt mixes (Potti [26]). This project also tried to enhance understanding the interactions between old binder and rejuvenators.

Moving to even more applied issues, EAPA (European Asphalt Pavement Association) edited an industry produced document including a state of the art in recycling of road materials with a wide overview of the currently available processes and data on the use of reclaimed asphalt in asphalt in 2001 and 2004. An update of those statistics is annually made in the EAPA document: "Asphalt in figures".

In terms of standardisation, the European Standard EN 13108, on bituminous mixtures and material specifications mentions the use of recycled materials with respect to binder and aggregate selection. Part 8 of this standard is dedicated to the description of materials from the recycling of old pavement mixtures or any other mix production surplus for use as components of mixtures in hot mix plants.

More recently, at the beginning of 2009, two European projects initiated by FEHRL, dealing with recycling in road materials, have been launched as part of the 7th Framework Programme. The first project, “Re-Road – End of life strategies of asphalt pavements [4]”, a collaborative programme led by Swedish VTI, is focussed on technical and environmental aspects of asphalt mixtures recycling, aiming to recycle asphalt mixtures at 99%, while avoiding down-grading, with special attention to asphalt mixtures containing polymer modified bitumen. The second project “Direct-Mat – sharing knowledge and practices on recycling of road materials in Europe [3]”, a collaborative, support action led by former LCPC (now IFSTTAR), aims to gather European practices on recycling of any road material back into roads. The main deliverable of this project is a Web database of these European practices.

Anticipating this trend, Task group 5 – Recycling of bituminous materials – of the RILEM Technical committee 206-ATB (Advanced Testing of Bituminous materials), has focussed its work on recycling of bituminous materials. In its framework, a field survey was carried out throughout Europe to make an overview of the current practices in terms of usage of Reclaimed Asphalt (RA). Several common research actions have been launched in order to address what was assumed by the group to represent key investigations to improve recycling of bituminous mixtures practices.

This chapter first presents the results of a survey performed on the practices used in Europe. This survey is considered a first report on the subject and could be usefully completed by the states of the art prepared in the framework of the Re-road and Direct-Mat European projects. Then, the chapter addresses the key issue of lab manufactured RA which is considered important for assessing recyclability of bituminous mixtures. To answer this question, common research has been launched on the basis of an interlaboratory test programme to set a relevant laboratory ageing of bituminous mixtures protocol. The choices of the tested protocol, the setting of the interlaboratory test programme as well as the results obtained, are presented in this chapter. Conclusions in the form of a recommendation for laboratory ageing of bituminous mixtures are given at the end of the chapter.

7.2 Use of Reclaimed Asphalt in Europe

7.2.1 Framework of Questionnaire

7.2.1.1 Information Sources

The core of this chapter is based on a questionnaire sent to local sales and technical representatives of the Total Bitumen European team amended by RILEM TG5 members and completed with information coming from other sources, such as the EAPA statement document, the FHWA European Scan Tour, 2000 report, interviews of some European experts and reactions on the draft of this report.

7.2.1.2 Countries

The countries involved in the survey were: Belgium, Czech Republic, France, Germany, Italy, The Netherlands, Nordic countries (particularly Denmark), Spain, Switzerland and the United Kingdom.

7.2.1.3 Questionnaire

The European field survey on the use of RA (Reclaimed Asphalt) in European countries was based on the answers to the following questions:

- Is it mandatory to use RA? Are there specifications?
- To which extent? Percentage of RA in a mix?
- What kind of technique is used? Cold recycling, hot recycling, foams?
- What type of fresh binder is added? Are rejuvenators used?
- What type of mix? What about Polymer modified Bitumen (PmB) mixes?

7.2.2 Responses and Discussions

Rather than showing the responses to the questionnaire country by country which would be long and tedious, the answers are summarised by themes in tables and bullet points showing common practices, discrepancies and trends (Planche [24]).

7.2.2.1 Regulations

Results are presented in Table 7.1, which reports the regulations in the various countries included in the survey. All the European countries have to use the European standard on RA EN 13108-8, but in some countries, there are also national requirements for the use of RA.

Discussion

Common Practice

There is no country yet with a national regulation on the use of recycled asphalt materials. However, most road administrations encourage the use of RA. In certain contracts, companies who can recycle will have an advantage.

Table 7.1 Regulations

Country	Must use RA	National requirements	Comments
BE	No	Regional: Flanders, Wallonia, Brussels Cap. Reg.	Flanders (F): Standaardbestek 250- SB250 v.2.2 Wallonia (W): RW99 G22213
CZ	No	Yes	Ministry of Transport specifications TP109
FR	No	Yes	Recommendations/guidelines
DE	No	No	Recommendations/guidelines
IT	No	No	ANAS agency ordering works of new highways – public body responsible for managing the national road network. Awaiting EN 13108-8 implementation
NL	No	Yes	Strong push from the administration to use RA
SE, FI	No	No	
NO	No	Yes	KFA ^a established in 2001 for managing RA aimed that 80% of the RA should be recycled in pavements within 5 years. Target well exceeded, every year after 2002, 90% or more RA having been recycled.
DK	No	Yes	High economic incentive to use RA when possible, as bitum. binders & aggr. for surface layers are imported goods. No specs but obligation to fulfil the road standards. RA can be added as long as the specs are met.
ES	No	Yes	Highway Administration clause 20 (cold) and clause 22 (cold) in PG4 document for Highway works
CH	No	Local	Due to the Swiss federal structure
UK	No	Yes	Highways Agency clauses 902 (hot) and 948 (cold) for Highway Works (SHW)

^aKFA: Kontrollordningen For Asfaltgjenvinning (The Information Centre for Recycled Asphalt)

Specific Situations

In some countries, there are national specifications such as in United Kingdom where the Highway Agency Clauses 902 for hot recycling and 948 for cold recycling are meant to facilitate the uptake of recovered materials. Also in the Czech Republic with TP109 from the Ministry of Transport and Spain with clause 20 and clause 22 in PG4 document for Highway works.

In Norway, the use of recycled asphalt is specified in clauses 622, 632 and 651 of the Håndbok 018. The clauses allow the use of both hot and cold recycled mixes. The use of cold recycled asphalt is allowed in the top layers and in the base layers of pavements under certain conditions (with an annual average daily traffic AADT < 3,000 and AADT < 5,000 respectively). Hot asphalt mixes that contain recycled asphalt can also be used in both the surface and the base layers of pavements with high traffic. However, the asphalt mix containing the recycled asphalt should meet the same requirements as the original asphalt mix.

In others, those specifications are at a regional level, such as in Belgium with three situations, Flanders (F), Walloon (W) and in the Brussels Capital Region, and Switzerland according to its federation structure.

In France, recommendations issued by the French Government Road Director insist on the need to systematically recycle at a low rate, and recently the target set jointly by the government and the road companies is to achieve, in 2012, a reuse in bituminous materials of at least 60% of RA collected each year. In 2008, the French Committee for Road Engineering edited guidelines for using hot mix asphalt standards, specifying the possible RA content as a function of the pavement course, the characteristics of the binder and the grading curve of the RA.

7.2.2.2 Recycling Techniques

Results are presented in Table 7.2, which reports on the techniques used, giving the maximum RA content authorised, according to the related layer in the new pavement, and to the technique used for mixing with fresh materials.

Discussion

Common Situation

Reclaimed bituminous materials are mainly reused in hot techniques. They may be used in the production of bituminous surfacings and base layers. The maximum amount of reclaimed bituminous material permitted is in general 10% in surface layers and 50% in all other layers. The EN 13108 standard recommends to adjust the binder type when considering more than 10% for surface layers and 20% for base layers. Additional performance requirements may be needed when the recycled content exceeds about 5–20% (in weight) depending on the country. The inclusion of 10% RA appears to be the norm. Most suppliers do not exceed 30% of recycled content due to manufacturing restraints within the plant. However, more modern mixing plants are now capable of handling 70% RA.

Cold or warm bitumen emulsion/foam mix technology can incorporate up to 100% recycled aggregate content. This can be used in surface, base and binder layers with in-situ/ex-situ processing techniques. Mixes with RA have to meet the same mix specifications as mixes with virgin materials. The RA content obviously depends on the amount of aged binder in the RA and the amount of fresh binder to be added, particularly its softness, but also on the hot mix unit equipment, such as drum-dryers, double barrel drum mixers, as described in the EAPA document. Additional performance requirements are needed when the recycled content exceeds about 5–20 wt.% depending on the country. The inclusion of 10% RA appears to be the norm. Most suppliers do not exceed 30% recycled content due to manufacturing restraints within the plant. However, more modern mixing plants are now capable of handling 70% RA. Cold or warm bitumen emulsion/foam mix technology can incorporate up to 100% recycled aggregate content. This can be used in surface, base and binder courses with in-situ/ex-situ processing techniques.

Table 7.2 Recycling techniques

Country	Hot technique	Max RA content in base/binder layers	Max RA content in surface layers	Cold technique
BE (F + W)	Mainly	50% binder homogeneous RA 20% binder heterogeneous RA 20% binder RA in EME ^a 25–40% when fresh bitumen pen 30 to 70 60% for softer grade	prohibited	20% in base layer
CZ			No	Foamed bitumen Emulsions and Cement
FR	Mainly	15% with no testing on national network. Up to 40%	Up till 10% with limited restriction and till 40% with restriction on the binder content	Emulsions or foamed bitumen
DE	Mainly	20% in binder course Up to 100% in base	20% (Direct-Mat. report states 100%)	Emulsions or foams Recommended for tar contaminated RA
IT	Mainly	Never above 50%, usually below 30%	No, except for special wearing course – below 20%	Mainly emulsions but foam increase Higher % RA allowed
NL	Mainly	50% in base layer 50% in binder layer 50% in DAC surface layers	50%	
NO	Mainly	No limit ^b	No limit ^a	Emulsions and foamed bitumen
DK	Mainly, but much goes to unbound application	5–10% with no testing (Direct-Mat report states 100% for base course and 30% for binder course)	5–10% with no testing (Direct-Mat. report states 30%)	Almost none
ES		10% with no testing 11–50%	Not permitted	100% RA in base layer. Emulsions

^a Enrobé à Module Élevé (high modulus asphalt)

^b If RA content > 10% in the wearing course or > 20% in the base layers, the qualities of the binder in the recycled mix should meet the binder specifications of the target mix. Described in Sect. 7.4

Mixes with RA have to meet the same mix specifications as mixes with virgin materials. The RA content obviously depends on the amount of aged binder in the RA and the amount of fresh binder to be added, particularly its softness, but also on the hot mix unit equipment, such as drum-dryers or double barrel drum mixers, as it is very well described in the EAPA document.

Specific Situations

German recommendations for the use of different types of asphalts are worth noticing:

- RA from Gussasphalt (mastic asphalt) can be recycled in Gussasphalt, due to particular aggregate gradation
- RA from surface layer can be recycled in surface layers or inter layers
- RA from inter layers, in interlayer and base layers
- RA from base layers, in base and sub-base layers

The RA is crushed and sieved for stockpiling in different grading for example 0/10 or 1/16 dependent on application. Some preliminary characterisation of the binder will determine the final application based on the Ring and Ball (RB) softening point. Above 70 °C, the binder is considered as too heavily aged and should not be reused at more than 10% in the final mix. Below 70 °C it can be used up to 30–50%, the limits being only the plant capacity and available stockpiles.

In Belgium different rules for the addition of RA are defined in the regional regulations (Flanders, Walloon and Brussels-Capital Region) and for different works (public, private works or works with a certificated mixture). For Flanders, there are no regulations for private works; so for those works, no maximum RA addition is defined. In case of public works in Flanders, the regional standard (SB250 v2.2) defines the maximum percentage of RA-binder allowed in the mixture. The use of RA is allowed only for base layers. Within these Flemish regulations two types of mixtures are considered: those in accordance to the standard EN13108-1 (mixtures following the fundamental method and table 2 of EN13108-1) and those defined within SB250 2.2 following the empirical method (in accordance with some aspects of the EN13108-1 and additional specifications on mixture design). For the first type of mixtures, only homogenous RA is allowed (no maximum is defined) and for EME the allowable percentage of RA-binder is limited to 20%. For the second type of mixtures the maximum allowed percentage of RA-binder is 50% when using homogenous RA and 20% when using heterogeneous RA, in case of hot addition (parallel drum). When the RA is added without preheating (minimum 110 °C) the maximum amount of RA-binder is set to 20% (Van den bergh [31]). In Flanders, RA is accepted as a secondary material since 1980s. Some studies showed that RA fulfils the requirements for materials to be used in contact with groundwater carrying soils. The criteria used are concentrations of hazardous materials and concentrations of leaking metals. The concentration of PAHs (Polycyclic aromatic hydrocarbons) is so limited so that tar containing asphalts are not allowed for hot recycling. For production control, extra testing on RA and continuous monitoring of RA is needed. RA has to comply

with the standard EN 13108-8: Reclaimed asphalt. For Belgium, extra requirements are regulated in TRA13 (Regulations edited by the certifying body COPRO). For example the binder of the RA must be identified. The maximum size of the aggregates is 63 mm. The RA is categorised into four classes (Van den bergh [30]). In (Van den bergh [31]) a brief survey of RA characteristics between 2002 and 2009 for Flanders is reported. For 2009 the mean penetration of RA is 20.3 [1/10 mm] ($s = 5.9$ [1/10 mm]). From an economical point of view, heterogeneous RA-batches are mixed together to homogenous RA-batches.

In Nordic countries, the use of RA will increase in the future. The Finnish Road association thinks that RA may account for 90% of the surfacing material of new roads in the future due to the problems of availability of suitable aggregates, in addition to the threat of oil shortage (Hugener [13]). This lack of good aggregates is partly caused by the extensive use of studded tires at winter time in Sweden, Norway, Finland, and Iceland. High trafficked road pavements often need recycling after about 5 years in service due to an undesired transversal profile (aquaplaning risk etc.).

In Denmark the situation differs due to the absence of studded tires. Surface courses normally die of old age due to fretting and ravelling etc., at the age of 12–17 years. The penetration of the recovered binder is then typically in the range 15–25 [1/10 mm]. This would imply a rather small amount of recyclable materials available. So the addition of RA stays in the range from 5 to 10% of the total mix, meaning very little effect of the variability of the hardened binder properties in the new mix. However, soft asphalt is added and, on the tertiary road network, an analysis in accordance with EN 13108-8 shows a rather homogeneous composition of the RA, with binder content around 4.5% and a penetration close to 50–60 [1/10 mm] which is quite close to what is normally produced for lower bituminous bases. Anyway, these limits in base, binder and surface layers can be pushed up, through RA pre-processing (crushing/sieving) where the contractors analyse the RA in accordance with EN 13108-8 and are then allowed to put in as much as they like “provided they can fulfil the requirements of the road standards”. Therefore, in base layers typically, RA is used as much as the asphalt plant is capable of, if pre-processed RA is available, usually in steps of around 15, 30, 40 and 60% depending on plant configuration. In some binder courses RA can be used, but hardly ever in case of heavily trafficked highways where crushed aggregates are preferred. In wearing course RA is very seldom used, indirectly because it does not match normal mix constituents. Gravel from glacial deposits, a predominant aggregate, can contain up to 5% frost sensitive particles. Although accepted in base layers, these particles are not allowed in wearing courses. It is not economically possible to perform selective milling of the old wearing course, so all RA is pre-processed into one 0/16 mm fraction, unfit for wearing course and in most cases also for binder course as the gravel is not sufficiently crushed to meet the requirements. The polymer content in RA is roughly 2–4% at this moment, so not worth separating. The produced polymer modified asphalts have not yet reached such a large extent that they would be subjected to recycling. That will change in 5–10 years.

In France the reuse of RA into Hot Mix has been practised since the 1970s. However, more engineering is coming in force now with the application of the EN

standards. The rules on how to reuse RA are based on its consistency and associated thresholds [1]:

- The binder content variability, below 1% allows recycling up to 40% while above 2% no more than 10% (0% for surface layer)
- The binder property variability looking at penetration value with minimum value of 5 [1/10 mm] and RB-softening point with maximum value of 77 °C
- The variability of grading
- The characteristics of aggregates mainly with regarding to the toughness (Los Angeles value)

Trend

The increased use of recycled and secondary aggregates in highway construction is a key policy objective of the Highways Agencies. Several countries are looking for 100% RA in mixes, in the long-term future, and all seek for using more recycled asphalt in road paving and particularly in new pavements, with as little as possible downgrading. There is often a tendency to use RA in unbound layers, but the trend is to reuse the RA at the highest possible application level.

7.2.2.3 Type of Mixes and Binders

Results are presented in Table 7.3 reporting on the types of mixes and binders.

Discussion

Common Situation

In terms of asphalt mixes, RA is generally used in dense mixes. However, countries such as the Netherlands even use RA in open porous friction courses.

In terms of binders, Europe wide, most added binders are normal paving grades selected to account for the hardening of the aged RA binders, according to classical blending rules on penetration and RB-softening point.

Germany, Switzerland and Czech Republic tend to use more and more PmBs as fresh binders instead of just regular plain bitumen in order to improve the mechanical properties of the RA included mixes, particularly regarding cracking resistance. The other purpose is to increase the RA content by bringing more quality to the final mix (Hugener [13]).

The use of rejuvenator is very occasional. There is no clear evidence of benefits, particularly in comparison with soft bitumen grades which are economically considered as more interesting.

Tar contamination: it is a major issue due to the PAHs (polycyclic aromatic hydrocarbons) contamination coming from some old pavements. Tar contaminated RA is even forbidden by law in the Netherlands and Belgium, and of concern

Table 7.3 Types of mixes and binders

Country	Type of mix	Added binders	Rejuvenator	Additional comment
BE (F + W)	Base coarse. PmB mix requiring further measurement	Normal grade	Scarcely used	Resulting Binder (aged & new) to meet specs of fresh binder. No tar
CZ		Normal grades PmB's (RC type)	Occasionally used	Tar contamin. RA restricted
FR	No problem w. PmB RA	Normal grades	Scarcely used	Tar contamin. RA restricted
DE	Gussasphalt SMA starting	Normal grades PmB's (RC type)	Scarcely used	PmB RC added to increase % RA in wearing & binder layers. Tar contamin. RA restricted
IT	PmB RA used by agencies segregating their own PmB	Normal grades	Scarcely used	
NL	All sorts including large quantities of porous asphalt (PA)	Normal grades with the log pen rule	Scarcely used – soft bit. instead	Healing factor to be considered. Tar contamin. RA forbidden. Strongly increasing RA from PA
NO	Recycling allowed for all the standard types of asphalt mixtures in Norway	Normal bit. grades w. log Pen rule. Softening pt. of added bit. is related to target soft. pt..	Scarcely used	
DK	–	Normal grades with log Pen rule	–	Evaluation of the ageing properties of the fresh binder
ES	All kinds except porous asphalt	Normal & rejuvenating grades. No PMB	Occasionally used	
UK	Mainly dense in binder course	Normal grades No use of PmB	Up to bit. suppliers	

particularly in Czech Republic, Denmark and Germany. In Switzerland, the PAH-content has to be determined before reconstruction of old roads, and should not exceed 5,000 ppm for new pavements with RA (Hugener [13]).

Hence, tar contamination was one of the main items of the SAMARIS project which came up with some recommendations in terms of procedure to identify the presence of coal tar in secondary component materials and recycling in the production of asphalt (Nicholls et al. [18, 19]).

Specific Situations

In the Netherlands the importance of the healing factor is acknowledged because of the high RA contents in mixes: after the VBW¹/Benelux Bitumen study (Jacobs et al. [14]), RA mix design should account for healing, which depends on the fresh added binder. Fresh binders are better healers than old ones. Contractors are currently discussing with the administration to decide on the healing factor. Also the availability of RA from Porous Asphalt (PA) with very low penetration value is a major problem in the Netherlands, due to the high amount of PA used as surface layer for highways since 20 years.

German recommendation on the aged binder quality: care to be taken with extracted binders with a RB-softening point higher than 70 °C. Those are not forbidden, but cautiously used.

France looks at the variability of the RA properties such as binder content, binder properties and aggregates. The binder properties thresholds are set at minimum penetration value 5 [1/10 mm] and maximum RB-softening point of 77 °C.

In UK the threshold to reuse RA into hot mix is set for the penetration value with minimum of 15 [1/10 mm] and additional testing on the aggregates for the reuse into surface layers.

Denmark: recommendation on the fresh binder quality with respect to its ageing potential. As an indicator of potential premature ageing, the Danish Asphalt Industry follows the viscosity index of binder aged in the Pressure Aging Vessel PAV (e.g. the viscosity ratio at 60 °C of PAV aged binder over the virgin binder determined by EN 12596).

Earlier experience showed that this “viscosity index” would have raised question marks about binders failing prematurely. Such an indicator is valid for pure or “normal” bituminous materials only. In case of modification, the industry is very well aware of the fact that any modification used now must not hinder or endanger the recycling potential of the material. This is to prevent from using particularly some metal compounds modified binders susceptible to evolve erratically during ageing (industry statement on the recycling of asphalt mixes and use of waste of asphalt pavements).

This point is of particular importance considering bituminous materials in Life Cycle Assessment (LCA).

7.2.3 Summary of European Practices

As a main and common tendency, the use of reclaimed asphalt is growing throughout Europe. However the use differs from country to country, even if there are similar usages and trends.

¹ VBW Vereniging voor Bitumineuze Werking (Dutch Road Builders Association).

This survey allows the following conclusions:

- In terms of regulations, the use of RA is generally not mandatory, but there is an increasing pressure from highway agencies to make it mandatory. In general, there are no national specifications, but rather recommendations or guidelines or even sometimes local specifications. Some countries have no obligation but specify the addition of RA in asphalt mixes. A European standard is in progress to normalise the use of RA.
- As far as techniques and use are concerned, the main techniques involve hot recycling, but cold recycling through either emulsion or foamed bitumen is growing and becomes prevalent in some countries.
- RA content limits, for hot recycling, are almost always below 50%, usually below 30% (except in the Netherlands), depending on the layer, whereas for cold recycling they can be as high as 100%. Ten percent is often the limit under which there is no need for testing the resulting mix.
- The main use is in base and binder layers, but new uses are occurring in surface layers for SMA and dense mixes.
- Cold techniques appear to have a huge potential in helping to increase the amount of RA, and noticeably the tar contaminated RA mixes
- Rejuvenators are not so much used due to cost effectiveness issues
- PmB's are getting more used, either as reclaimed modified mixes or as fresh PmB in place of rejuvenators and in emulsion for cold recycling
- In terms of HSE (Health, Security and Environment), the risks when using old pavements should not be forgotten. This issue was addressed in the SAMARIS project which spent a substantial amount of effort assessing the detection of tar and more particularly PAH (Polycyclic Aromatic Hydrocarbon) products coming from coal tar, present in old pavements. This must be considered in the methodology of using RA.

The economics of using RA were not specifically addressed in this paper. They are of course at the heart of the problem. A study from the Netherlands showed that RA, although dependant on demand and supply, generally induces a 10% reduction in asphalt price. Asphalt plants producing mixes with RA normally use 3.5% binder by volume, which is up to 50% less bitumen. This would certainly be a major incentive in the long-term to use more RA in mixes, but there is still a long way to go before routinely using 100% of RA for all the technical and HSE reasons described above.

The reuse of RA at a medium or high rate is usually correlated with an increase of energy use as the virgin aggregates have to be heated to a higher temperature to offset the warmer RA introduced and also still the need to dry/soften the RA binder at 120–130 °C. This result leads to an increase of GHG at the mix plant that is often balanced in reducing GHG (GreenHouse Gas) emissions and energy use at aggregate extraction and bitumen supply.

Last but not least, this synthesis will have to be updated continuously as developments evolve very rapidly in the field of recycling. It will probably be updated soon after the continuation of the work of the RILEM TC SIB/TG5 working group.

7.3 Development of a Laboratory Ageing Protocol for Bituminous Mixtures

7.3.1 Introduction

Among the key points related to hot recycling of bituminous mixtures such as homogeneity of RA, recyclability of bituminous mixtures and mix design of mixtures containing RA, TG5 has decided to focus its work on recyclability of bituminous mixtures. This issue raises the question of reliable laboratory production of RA and as a consequence of a relevant methodology in laboratory to age bituminous mixtures.

The benefits of having a laboratory procedure that can produce consistent “RA-products” are twofold; it allows investigations on special recycling binders, for example that can regenerate aged asphalt mixture. It also helps to predict ageing of newly-developed products over time in the mix itself and then determine its ability to be reused again.

Such a methodology does not exist at the international level although some protocols have been proposed in several countries. In the framework of TG5, a first review of the existing protocols has consequently been first undertaken. This review has been followed by the proposal of a common protocol, which has been tested in the frame of a Round Robin Test (RRT) to assess its relevance.

7.3.2 Inquiry on Current Practices in Laboratory Ageing

Previous studies have been carried out to define a laboratory procedure that can predict the future ageing of asphalt mixtures (Porot and Bobrisow [25]).

The NCHRP programme A383 (Bell [6]) has defined a laboratory ageing procedure for asphalt mixtures. The AASHTO R302 method is based on that procedure and it consists of:

- A short-term ageing step in which the loose mixture is aged in a forced-air draft oven for 4 h at 135 °C. This is supposed to represent an average level. Actually, mixtures may undergo very little ageing or considerably more ageing.
- A long-term ageing step in which compacted mixture specimens are aged in an oven for 5 days at 85 °C.

The process can be accelerated to 2 days by using 100 °C, but the NCHRP A383 (Bell [6]) report stresses that this could lead to specimen damage. This method is still a subject for research. The limitations of the protocol are related to the correlation with field data and in the differentiation by mixture type and thickness (Raghavendra et al. [27]).

In Japan, research has also been conducted to clarify the effect of ageing for asphalt concrete. A compacted sample was aged in an oven at 70 °C for specified periods (Hachiya et al. [12]). In Europe, the University of Nottingham has developed a specific test called Saturation Ageing Tensile Stiffness (SATS) to assess the ageing and moisture sensitivity of asphalt mixtures (Airey et al. [5]).

However, all these methods use compacted samples. To produce materials for further use “as RA”, compacted samples must be reheated to become loose materials. This is one of the reasons why these methods have to be modified for the purpose of future recycling studies.

In 2006 a first experimental test for a Belgian ageing method on loose mixes was conducted by Artesis Hogeschool (Van den bergh [30]). Several asphalt mixtures were aged in an air ventilated oven during a short-term ageing period (3 h at 130 °C) and a long-term ageing period at 90 °C. The binders extracted from the aged mix were characterised using a DSR (Dynamic Shear Rheometer), penetration, RB-softening point and Fourier Transform Infrared Spectroscopy (FTIR). Binder ageing is observed by an increase of RB-softening point, G^* , CO- and SO-indices (see Sect. 7.5). After 7 days a plateau is reached for both chemical indices. This level is similar as for extracted RA-binders. The rheological ageing indices, as G^* and RB-softening point, however do not display such a plateau. The ageing process continues by means of binder hardening. After 7 days the penetration is decreased from 47 [1/10 mm] to 19 [1/10 mm]. After 10 days 16 [1/10 mm] is achieved, which is much lower than the mean RA-binder around 20 [1/10 mm] for Flanders. For pure binder ageing (short-term and long-term Rotating Cylinder Aging Test, RCAT) the chemical index SO (Iso) shows a plateau after 7 days while for the CO index (Ico) ageing still increases but is still lower than the maximal obtained value for RA. After 7 days, both indices have similar CO and SO indices as for RA-binder. Both rheological as chemical ageing indicators show that ageing of pure binder by RCAT or binder in artificial aged asphalt mixtures during 168 h lead to an aged binder with 40% of the initial penetration values (same as for other binders) and with Ico and Iso values comparable with RA-binders (see Van den bergh [31], chapter 7).

In addition, previous works done in this area have shown that the correlation with field data is extremely difficult. Such correlation is affected by the location of the road, sun exposure, climate, mixture type and void contents.

In the framework of the Re-Road project ([4]), four ageing methods on loose mixes are being compared. These methods are talked about in the following section. The RILEM TC 206-ATB TG5 method is actually a clone of the AASHTO R30-2 ageing method, a RTT has been performed to assess this method, mainly in

Table 7.4 Comparison of four different ageing methods, in the framework of a Re-Road study

	RILEM ageing	PAV ageing	BRRC ageing	UV ageing
Sample type	6 cm height	1 cm height	3 cm height	3 cm height
Oven type	Air draft	Pressured air	Air draft	Air draft + UV
Short-term ageing	135 °C, 4 h	135 °C, 4 h	135 °C, 1.5 h	135 °C, 1.5 h
Long-term ageing	85 °C, 9 days	100 °C, 2.1 MPa, duration to be set	60 °C, 14 days	60 °C, 14 days

terms of reproducibility. It is described in the next sections. The PAV method was initially used to get bitumens aged. It has been adapted to get loose mixes aged. The BRRC (Belgian Road Research Center) method (Piérard and Vanelstraete [21]) is comparable to the RILEM method, but the temperatures for short-term and long-term ageing and thicknesses of specimens are different. Finally, the UV method is quite similar to that of BRRC, with added UV radiation. This method is still being developed. The main characteristics of the four methods are described in Table 7.4.

A first comparison of the different methods has been performed, while all the results are not available yet. All these methods have advantages and disadvantages. If we consider the RILEM ageing method as the reference method, compared to the PAV ageing, it seems easier to perform, because the apparatus needed is simpler. However, PAV ageing may be more reproducible; the PAV apparatus is part of the EN-14769 standard “Accelerated long-term ageing conditioning by a Pressure Ageing Vessel”. Thus, whatever the laboratory, the apparatus is supposed to be the same. This is not the case for the ovens used for the ovens used for the RILEM ageing, especially. . . RILEM ageing, especially in terms of air drafting, that may impact ageing. Another point to compare is the quantity of material that can be aged at once. In the PAV, only a small quantity of material (around 1 kg) can be aged, which may strongly limit the number of tests on aged specimens, due to a low quantity of aged material. The BRRC method involves ageing less material at once, due to a lower thickness of the aged samples. The BRRC method exhibits a shorter “short-term ageing”, and a longer “long-term” ageing. The main point of this method is the low temperature during the long-term ageing, which is supposed to keep the sample below the RB-softening point temperature. This low temperature may prevent segregation inside the material, due to too soft bitumen, and problems of polymer fusion when polymers are used. As said previously, the main practical disadvantage of this method is the low height of samples. The UV method is based on the BRRC method, while imposing an added UV radiation. The main interest of this method is to take into account the influence of UV on ageing. It seems, a priori, to be more representative of conditions on site. The main disadvantage of this method is the need of an UV oven, which can be expensive. For a good control of the ageing, a good combination between the parameters (temperature inside the oven, air draft and UV radiation level) is needed. In such ovens, only low quantities of materials can be aged in one time.

7.3.3 *Various Ways to Age Mixture*

From the above survey on the different laboratory ageing protocols, it is worth to note the key parameters that may differ and their suitability:

- Compacted vs. Loose material
- Ageing cycle as a combination of temperature and time
- UV effect
- The use of air pulses or oxygen pulses

Certainly the first parameter is the state of the sample. Here, two approaches are in use. The first approach is to use compacted samples as in the field and then to age regardless the real conditions. This method fits well for determining the end-characteristics of asphalt mixtures on the road. However, the ageing process through oxidation, heating, or UV the testing conditions are not homogenous within the sample. Recovering binder would need further operations that may affect the components. The second approach is to use loose material. This method is certainly more homogenous in terms of ageing the whole mixture, especially the binder. However, with such method, directly determining the mixture characteristics afterwards is not possible. It is worth to point out that as well as in the case of compacted sample, the shape and size of the sample affect the final results and their variability.

With regards to ageing cycle, a consensus has to be found between the temperature and the duration, in order to get time-scale which is acceptable for the laboratory. Different experiences have shown that a temperature below 100 °C is the best practice not to move too far from the real field conditions. The duration is then determined according to the targeted ageing degree. The pressure may be included here as a combination of temperature and pressure.

The UV effect certainly plays a role. However, until now, only a few investigations have been made on this feature. One thing that is agreed is that the UV effect is only sensible on the surface of the pavement and does not penetrate in the whole material matrix. The question remains open for thin porous asphalts layers, with high void contents. This point is definitively more relevant for top layer materials.

Lastly the possibility to have a flow of air or oxygen in order to stimulate and accelerate the oxidation may affect the results. However again in the field this is not true for the full depth layers (see Table 7.5).

A last point that was not addressed so far is ageing through cold cycles. However, for the purpose of the reuse of RA, it is well known that the ageing process usually ends with a hardening of the bituminous binder through oxidation and heating cycles.

The RILEM protocol was established on this analysis.

Table 7.5 Different parameters to classify ageing methods

Parameters	Options
State of sample	Compacted sample: Suitable for final characterisation of asphalt mixture (rutting, stiffness, fatigue, . . .) Loose material: Suitable to recover and characterise the components
Ageing cycle	Temperature: Should be kept below 100 °C Duration: Target laboratory scale
UV	Not enough data to address the real effect of UV, mostly relevant for surface layers
Air/Oxygen	May accelerate the ageing but should be match laboratory facilities and relevance to the real field conditions

7.3.4 *Proposal for a Laboratory Ageing Protocol for Bituminous Mixtures*

The goals of this testing were:

1. To provide a protocol that can produce a well-controlled RA-like material in the laboratory.
2. To understand the kinetics of ageing as a function of time.
3. To determine the variability of such a test method.
4. To compare/correlate if possible with other ageing approaches (on binder) or with field data.

Based on the previous literature review and expertise, the test protocol has been chosen by RILEM TC 206-ATB TG5. According to the analysis above and the defined targets, a protocol for “loose-materials” was selected.

The protocol consists of heating a bituminous mixture in an oven, in order to oxidise the material. A loose mixture is used to obtain materials aged more homogeneously. The size of the samples to be aged has been well defined in terms of height and surface to limit size effects during the oxidation process. UV ageing during service life has not been considered as it only affects a very thin section of the surface layer (Wu et al. [32]; Durrieu et al. [10]). Also with regards to air/oxygen flows, it was not considered here in order to comply as much as possible with laboratory oven facilities. For this reason, a classical heating oven can be used.

The ageing protocol has been divided into short and long-term phases after manufacturing (heating and mixing), in order to simulate ageing due to transportation and laying, and ageing on site during service life respectively:

- For short-term ageing, the loose mixture is aged 4 h in an oven at 135 °C.
- For long-term ageing, the loose mixture is aged 9 days in an oven at 85 °C.

The effect of ageing on the bituminous mixture will be assessed through measurements on the recovered binder at different steps of the protocol. The results

will be compared to classical ageing tests performed on binders. Empirical rheological (penetration and RB-softening point), fundamental rheological (complex modulus, phase angle) and chemical (oxidation levels) characteristics will be investigated. In the future, it is planned to compare these results to the in situ material properties after long-term ageing on site.

A Round Robin Test (RTT) has been set up to assess the feasibility of this protocol as well as its relevance regarding binder and in situ ageing. The results will allow the construction of a final methodology to manufacture RA in laboratory in a relevant way, after comparison with field recovered aged material from the same asphalt mixture.

The target was to answer to the following questions:

- Is this test protocol easy to perform in various laboratories?
- How relevant are the obtained results, in term of repeatability and reproducibility?
- Is it effectively possible to observe an ageing effect in laboratory?
- How are the ageing kinetics over time?
- Are there any limits or thresholds, within the ageing process?

7.4 Organisation of Round Robin Test

7.4.1 *Participants*

The list of the nine participating laboratories is presented in Table 7.6 and acronyms in alphabetical order.

7.4.2 *Materials*

A plant production mix and a laboratory mix made from the same mixture design and components are tested in each laboratory in order to make a comparison between both manufacturing methods. Three batches are provided in the test program:

- lab-mix: 36 kg of asphalt is mixed in each laboratory with a target temperature of 165 °C.
- Lab-mix 2, similar to lab-mix 1. It was decided to use these materials afterwards to improve the test protocol, if necessary.
- plant-mix: 36 kg of an asphalt mixture (composition and materials the same as lab-mix 1 and 2) manufactured in an asphalt plant is reheated at 110 °C in each laboratory for 4 h in closed buckets. It was assumed that this reheating had no influence on the ageing of the material.

Table 7.6 List of the participating laboratories (Alphabetical order)

Name (Acronym)	Country
Artesis University Antwerp (AHA) Delft University of Technology (DU)	Belgium Netherlands
Danish Road Institute (DRI)	Denmark
ISBS	Germany
IFSTTAR (former LCPC)	France
LRPC-Aix	France
NTEC	United Kingdom
SINTEF	Norway
TOTAL	France

Note: Artesis and Delft University of Technology co-operated for the round robin as one laboratory (AHA)

Table 7.7 Components and characteristics of the material tested

Component	Proportions (%)
0–2 mm Quarry Brefauchet	34
2–6.3 mm Quarry Brefauchet	16
5.6–11.2 mm Quarry Brefauchet	49
Fillers (recycled fines)	1
Pure binder 35/50	5.7 (of the mix)

All materials for lab-mix 1 and 2, and the mixture for plant-mix were delivered to each laboratory in 2008. The mix composition is given in Table 7.7. For the plant-mix 750 kg has been sampled in the paver and for the laboratory-mixture (lab-mix 1 and 2) 2,950 kg aggregates and binder have been sampled from the mix plant. Homogenisation of aggregates was necessary to make homogeneous samples for each laboratory.

All materials for lab-mix 1 and 2, and the mixture for plant-mix were delivered to the laboratories in 2008.

7.4.3 Program of the Round Robin Test

7.4.3.1 Ageing Protocol for RRT

The final mixture ageing protocol chosen for the RRT takes into account three steps which have to be consistent for all laboratories. These included the production of the mixture, the ageing of the mixture and the binder extraction and recovery. This last step is necessary only to evaluate the binder ageing in the developing phase of the ageing procedure but will not be required after the approval of the protocol.

The flow chart displayed in Fig. 7.1 presents the steps of the tests.

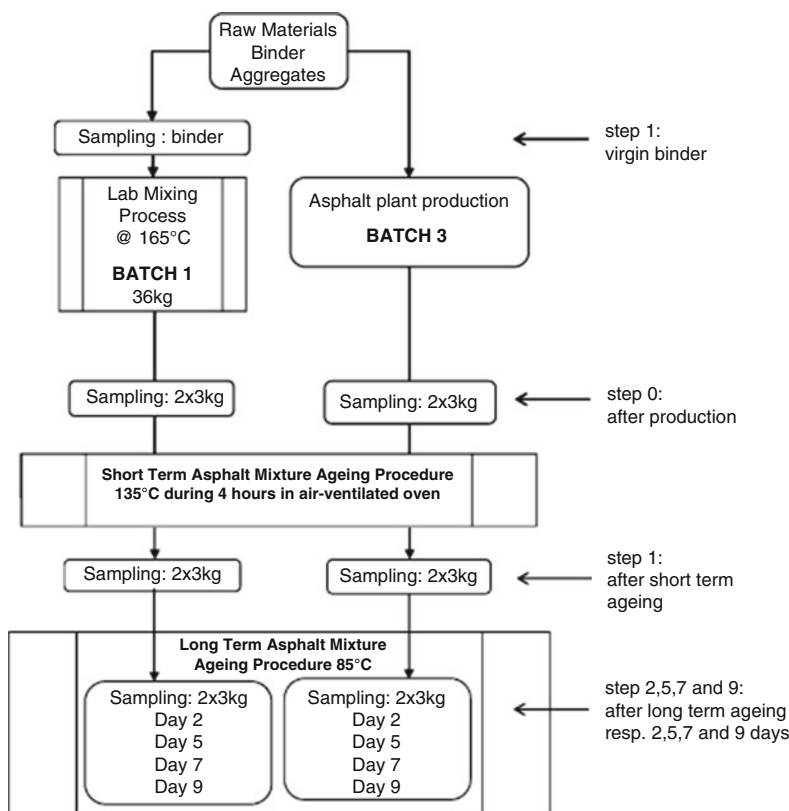


Fig. 7.1 Final mixture ageing protocol, with associated steps

7.4.3.2 Protocol Mixing Process (EN58, EN12594, EN12697-35)

The European Standards EN58, EN12594 and EN12697-35 are taken into account for the mixing process, in particular the aggregate and binder temperatures, mixing times and the way of adding the different components. For the binder B35/50 and a mixing process according to EN12697-35, the following settings are chosen:

- Target temperature of the mix: 165 °C
- Aggregate temperature during min. 8 h: 165 °C
- Binder temperature at maximum 165 °C
- Aggregates (first the stones, fines and filler) are put into a bowl and pre-heated up to the mixing temperature (165 °C)
- First Mixing Period: 30 s
- Binder addition (by weighing)
- Second Mixing Period: 2 min
- Sample 2×3 kg for further binder testing



Fig. 7.2 Metal box with an internal moveable flank (TU Delft)

Note 1: Where one batch, corresponding to a big quantity of RA, would need to be manufactured in several steps, the asphalt mixture would be temporarily stored in a closed box red at 165 °C until all material is ready. Before starting the ageing procedure, the different batches are mixed together for homogenisation.

Note 2: When the asphalt mixture has been manufactured in the asphalt plant, the mixture is reheated at 110 °C during 4 h in closed buckets. Afterwards, the mixture is homogenised. It is assumed that this does not affect the ageing of the material.

7.4.3.3 Ageing Facilities

The loose mixture is placed in an open box in an air ventilated oven.

The dimensions of the box are not defined, although it must be possible to put the asphalt mixture at a constant height of 5–6 cm during the ageing process. The oven must be equipped with a fan in order to circulate air in the oven and ventilate continuously the volume with fresh air during the ageing process.

A metal box with dimensions 30 cm × 50 cm × 8 cm (max. filling height 6 cm) is appropriate to age 15 kg of loose mixture. If during the ageing process intervening samples are taken, e.g. for periodic monitoring of the asphalt ageing process, the box must be equipped with a system to keep the thickness of the loose mix at a constant height of 5 to max. 6 cm. The boxes have one or two moveable sides (flank) in order to keep a constant height (see Fig. 7.2). For practical reasons,

Table 7.8 Sampling calendar

Wed	Thu	Fri	S	S	Mon	Tues	Wed	Thu	Fri
- Mixing		Stirring			Stirring		Stirring		Stirring
- Sampling (2*3 kg)		Sampling (2*3 kg)			Sampling (2*3 kg)		Sampling (2*3 kg)		Sampling (2*3 kg)
- Short-term									
- Sampling (2*3 kg)									

it is also possible to put wooden or metal boards in the box to decrease the box volume.

The ageing procedure includes two main phases: a short-term ageing period and a long-term ageing period.

7.4.3.4 Sampling Plan and Ageing Protocol for RRT

After production (lab-mix 1 and 2) or reheating up to 110 °C (plant-mix) short-term ageing and subsequently long-term ageing are conducted. For this, the asphalt mixture is placed into two boxes each with a surface of 0.15 m².

Two samples of 3 kg, designated a and b (each from one box), are taken after each step: after production, after short-term ageing and after 2, 5, 7 and 9 days of long-term ageing. The binder is extracted from all these samples and recovered for further analysis. Penetration, RB-softening point, DSR frequency sweeps and FTIR tests are conducted. After each sampling step, the movable flank of the box is moved in order to keep the height of the loose mixture constant.

In Table 7.8, an example of a testing calendar for each batch is given.

The temperature in the oven is set to 135 °C. Before starting the short-term ageing, the temperature must have reached 135 °C for 15 min at least, to be considered stable.

The mixture is heated to 135 °C in closed buckets before starting the short-term ageing.

Short-Term Ageing Procedure and Sampling

For the short-term ageing process, the mix (30 kg divided in two boxes) is placed in an air-draft ventilated oven for 4 h at 135 °C. Each hour, the material is stirred for 1 min and placed back into the oven. This stirring action is only for homogenisation. After this short-term ageing, the mixture is homogenised by stirring and 2 × 3 kg is sampled (sample a and sample b).

Table 7.9 List of laboratories

Group 1	Group 2
Lab 01	Lab 07
Lab 04	Lab 08
Lab 05	Lab 09
Lab 06	Lab 10

Long-Term Ageing Procedure and Sampling

Twenty four kilogram of loose cured material is mixed and divided into the two boxes. These two boxes are placed in an air-ventilated oven at 85 °C. After 2, 5, 7 and 9 days (e.g. short-term ending time $+48 \text{ h} \pm 1 \text{ h}$) the mixtures are first stirred (for homogenisation before sampling) for 1 min. After this stirring 3 kg mixture of box 1 and 3 kg of box 2 are sampled.

One laboratory (lab 09) performed the ageing test by stirring the mixture after sampling.

For convenience, the results for each step of the ageing process will be numbered in the chapter according to the step numbers indicated in Fig. 7.1.

As well as ageing of loose asphalt mixture, for further comparisons, the binder which is used in the asphalt mixture is also aged according to two ageing procedures:

1. RTFOT and PAV and,
2. RCAT short-term and long-term ageing.

These aged binders are also evaluated with penetration, RB-softening point, DSR and FTIR.

7.4.3.5 Binder Test Methods

In this project, each laboratory was invited to perform a full or partial testing program. In the framework of this RTT, two groups were made in order to take into account the influence of sample storage at ambient temperature between ageing step, recovering step and testing step on the rheological parameters and the ageing level of binder. In Table 7.9, these two groups are shown. The laboratory numbering is random.

- Group 1 performed binder recovery and binder tests following the chronological order of the ageing protocol (after manufacture, then after short-term ageing, then after 2 days with long-term ageing ...).
- Group 2 performed binder recovery and binder tests in reverse order (first, after 9 days with long-term ageing, then after 7 days with long-term ageing, then 5 days with long-term ageing ...).

From each sample of 3 kg mixture, the binder is extracted according to EN12697-1 (continuous flow centrifuge) and recovered for further testing using a

rotary evaporator according to EN 12697-3. Each laboratory used their usual solvent and equipment for convenience. No analysis was possible on the influence of the solvent and the type of apparatus used although this is expected to have an influence on the scattering of the results.

- P + RB: Penetration at 25 °C according to EN1425 (small cups) and Softening point (Ring & Ball) according to EN1426
- DSR (EN14770): frequency sweeps (G^* and phase angle) from 0.1 to 10 Hz in 11 equally spaced logarithmic frequency steps (0.1; 0.158; 0.251; 0.398; 0.631; 1.000; 1.585; 2.512; 3.98; 6.31; 10 Hz) at -5, 5, 15, 25, 35, 45, 55 °C or other in case of a specific standard; but a master-curve generation at 15 °C has to be possible. (10 rad/s = 1.59 Hz). If possible, the single point measurement can be integrated in the frequency sweep at 25 °C.
- FTIR (calculation of indexes according to LCPC testing method no 69: appendix A)

One laboratory also performed binder ageing according to RCAT-short-term and RCAT-long-term ageing device. Three laboratories performed Rolling Thin Film Oven Test (RTFOT) and two laboratories PAV.

7.5 Results of Round Robin Test

A summary of the received data is given in Table 7.10. The letters a and b correspond to the two samples described in Sect. 7.4.3.4.

7.5.1 Penetration and RB-Softening Point Results

Eight participants have performed penetration and RB-softening point tests for the lab-mix corresponding to laboratory manufacture. For the plant-mix, eight participants have reported penetration and RB-softening point. From these, one participant conducted the ageing of the plant-mix without the short-term ageing, in order to study the influence of this short-term ageing. Only one laboratory has performed the ageing procedure and tests on the lab-mix 2.

At this time, there is a limited statistical analysis of the raw data of penetration and RB-softening point. The mean of all results have been calculated.

A single result was rejected (outlier) when its value was beyond two times the s-interval. This “s” is calculated and interpreted as the estimation of the standard deviation of a random test with a Gaussian distribution. Note that this value may not be interpreted as the standard deviation “sigma” of a whole population.

In the following figures, it must be noted that the scale on the x-axis is stepwise and does not show the real time scale. The period between -1 (virgin) and 0 (after production of the mixture) refers to the sample preparation (heating of the

Table 7.10 Summary of received results for analysis

Lab	Lab-Mix 1			Plant-Mix			Lab-Mix 2			Original			RTFOT			PAV			RCAT		
	P + RB	FTIR	DSR	P + RB	FTIR	DSR	P + RB	FTIR	DSR	P + RB	FTIR	DSR	P + RB	FTIR	DSR	P + RB	FTIR	DSR	P + RB	FTIR	DSR
1	ab	ab	ab	ab	ab	ab	ab	ab	ab												
4	ab	ab	ab	ab	a	ab															
5	a	a	a	a	a	a				a	a		a	a		a	a				
6	ab	ab	ab	ab	ab	ab															
7	ab	ab	ab	ab	ab	ab				a	a	a	a	a	a	a	a				
8	a	a	a	a	a	a				a	a								a	a	
9	ab	ab	ab	ab	ab	ab				a	a		a	a		a	a				
10	ab	a	ab	ab	a	ab															

a and b correspond to the two samples of 3 kg taken from each box submitted to the ageing protocol (see § 7.4.3.4.)

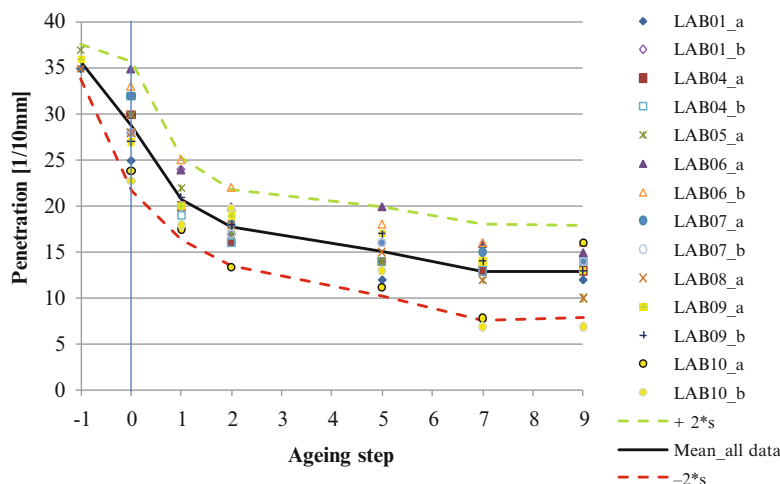


Fig. 7.3 Penetration Lab-Mix 1

materials: minimum 8 h) and the mixing period (150 s). The period between 0 (after production) and 1 (after short-term ageing) lasts only 4 h and for the steps afterwards the scale is expressed in days.

7.5.1.1 Penetration and RB-Softening Point – Lab-Mix

Figure 7.3 shows all results for the penetration tests (sample a and sample b) performed by the eight laboratories without the rejection of outliers and the interval considering two times s of the mean. Laboratories 5 and 8 have tested one sample only.

Ageing of the mixture is observed as a decrease in penetration. In general, the results show a parallel shift between laboratories throughout the ageing test e.g. Lab 10 shows a continuous negative offset for the penetration; lab 6 shows a positive offset from the mean.

The mixing process and short-term ageing (from step 0 to 1) can be distinguished from long-term ageing (subsequent to step 1) by the steep decrease of the results.

In Fig. 7.4, the RB-softening points are shown. Laboratories 5 and 8 have tested one sample only. A parallel shift-factor throughout the ageing test can be observed between the laboratories. Ageing of the mixture is observed as an increase of the RB-softening point.

7.5.1.2 Penetration and RB-Softening Point – Plant-Mix

Eight participants have performed penetration and RB-softening point tests for the plant-mix, corresponding to plant-mix manufacture. Figure 7.5 shows all results for the penetration tests (sample a and sample b) without any statistical rejection of

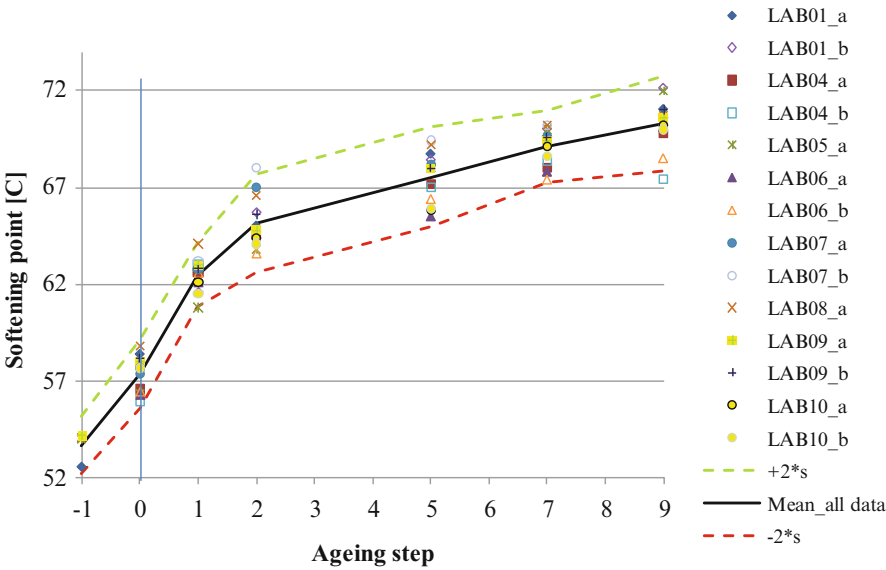


Fig. 7.4 Softening point Lab-Mix 1

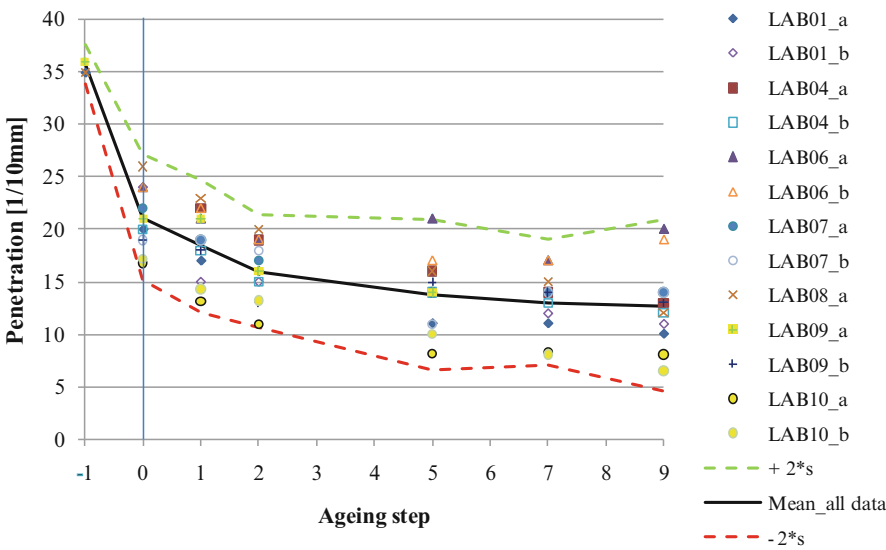


Fig. 7.5 Results of penetration tests for plant-mix

outliers. Laboratory 8 has tested one sample only. Lab 5 aged the plant-mix without short-term ageing. The results are not taken into account for the statistical analysis and not plotted in the figure. Lab 10 shows a continuous negative offset for the penetration; labs 6 and 8 show a positive offset from the mean.

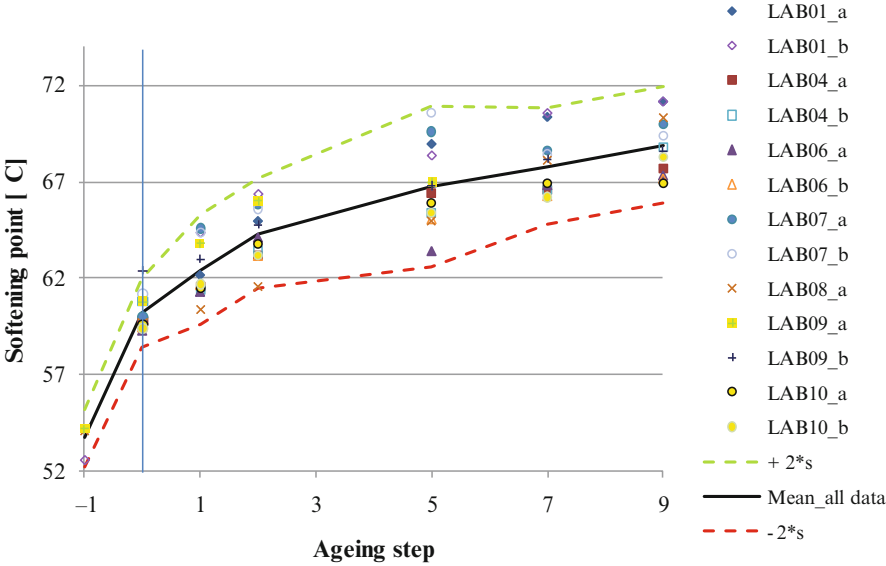


Fig. 7.6 Results of R&B tests for plant-mix

Ageing of the plant-mix is observed as a decrease in penetration values. Short-term ageing can be distinguished from long-term ageing (subsequently after step 1) since step 0 to 1 lasts 4 h and step 1 to 2 lasts 48 h.

For the plant-mix the penetration values of the samples at step 0 are lower than for the lab-mix which could indicate a higher ageing rate of the binder caused by the production in plant compared to laboratory production; the mixture is taken from the paver. Starting from step 2, the slope for the penetration curve of both mixtures are similar.

In Fig. 7.6 the RB-softening points are shown.

7.5.1.3 Penetration and RB-Softening Point Analysis for Lab-Mix and Plant-Mix

For both mixtures, ageing decreases the penetration and increases the RB-softening point. The analysis of the individual results shows that the initial value of the lab-mix is higher and lower for the penetration and RB-softening point respectively than for the plant-mix. After long-term ageing both mixtures show a similar penetration respectively 12.9 and 12.7 [1/10 mm].

In Table 7.11 and in Table 7.12 a summary is given of the data for penetration and RB-softening point of the binders from the lab-mix and the plant-mix respectively during the ageing process. In these tables the mean of all samples (samples a and b), the number of samples n and the standard deviation are shown.

Table 7.11 Penetration and RB-softening point data statistics for lab-mix: all data

		Penetration			RB-softening point		
		Mean	n	s	Mean	n	s
Ageing step		[1/10 mm]		[1/10 mm]	[°C]		[°C]
Virgin binder	−1	35.8	4	1	53.8	4	0.8
After mixing	0	28.7	14	3.5	57.4	14	0.9
After ST	1	20.8	14	2.2	62.5	14	0.8
After 2d LT	2	17.7	14	2.1	65.2	14	1.3
After 5d LT	5	15.1	14	2.4	67.6	14	1.3
After 7d LT	7	12.8	14	2.6	69.1	14	0.9
After 9d LT	9	12.9	14	2.5	70.3	14	1.2

Table 7.12 Penetration and RB-softening point data statistics for plant-mix: all data

		Penetration				RB-Softening point			
		Mean	n	s	Lab 05	Mean	n	s	Lab 05
Ageing step		[mm/10]		[mm/10]	[mm/10]	[°C]		[°C]	[°C]
Virgin binder	−1	35.8	4	1		53.8	4	0.8	
After mixing	0	21.2	13	3.0	16	60.2	13	0.9	64.2
After ST	1	18.4	13	3.2		62.5	13	1.4	
After 2d LT	2	16.0	13	2.7	14	64.3	13	1.4	66.4
After 5d LT	5	13.8	13	3.6	12	66.8	13	2.1	68.8
After 7d LT	7	13.0	13	3.0	12	67.8	12	1.5	70.4
After 9d LT	9	12.7	13	4.1	11	68.9	12	1.5	70.2

For Lab 5 the mixture is aged without a short-term ageing procedure. The other laboratories have performed the ageing protocol by a short and long-term ageing. The results of Lab 5 are not taken into account for the calculation of the mean and the standard deviation.

The plant-mix was manufactured for all laboratories at the same time. This means all laboratories have the same mixture for ageing step 0. The results of the binders extracted at step 0 (production) show a smaller standard deviation than for the lab-mix. After the short-term ageing process the deviations become larger.

In this RTT all binders (excluding stage -1) are extracted and recovered from different mixtures: the mixtures are manufactured and aged in different laboratories. Analysis was carried out by means of a normalisation at step 0 (value = 1) as shown in Fig. 7.7 for the normalised penetration values and in Fig. 7.8 for the normalised RB-softening points of the binder extracted from the lab-mix. With this normalisation process the ageing process starting from manufacture (step 0) can be compared at the different laboratories. Since no reproducibility interval is reported for the binder characteristics after extraction and recovery during an ageing test, it was agreed by the project partners to reject outliers defined by a deviation of two times the estimated standard deviation and recalculate the mean-values for

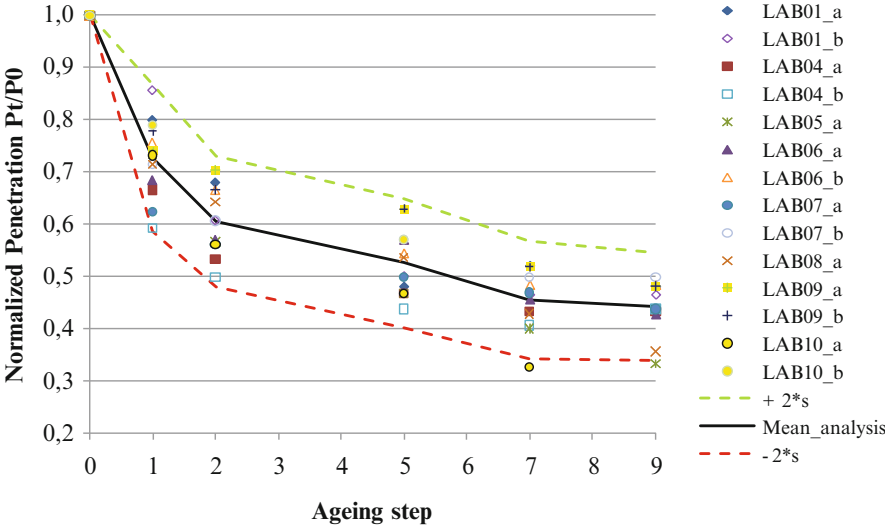


Fig. 7.7 Normalised penetration values for Lab-Mix

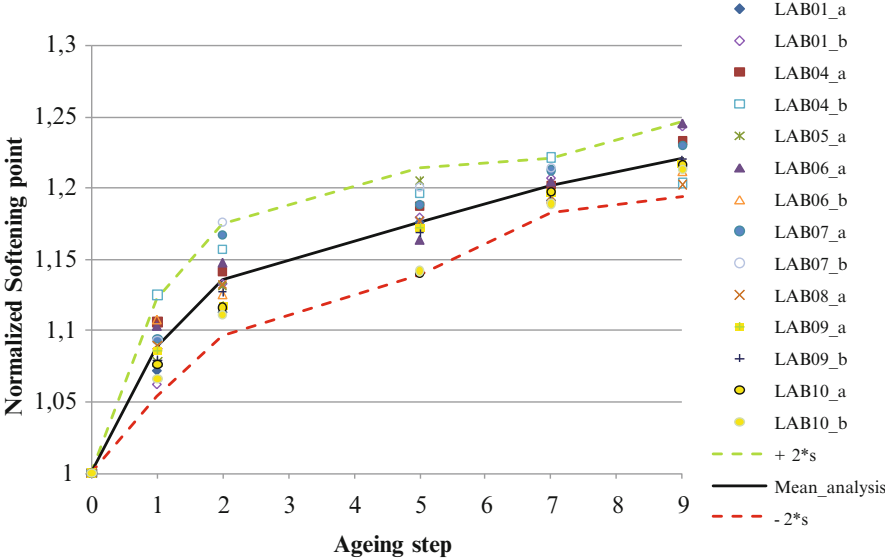


Fig. 7.8 Normalised results for RB-softening points of binder from lab-mix

the lab-mix and the plant-mix. The following figures consider the normalised values and the outliers are rejected and the mean is re-calculated.

In the Figs. 7.9 and 7.10, this analysis is done also for the plant-mix respectively for the penetration and RB-softening point.

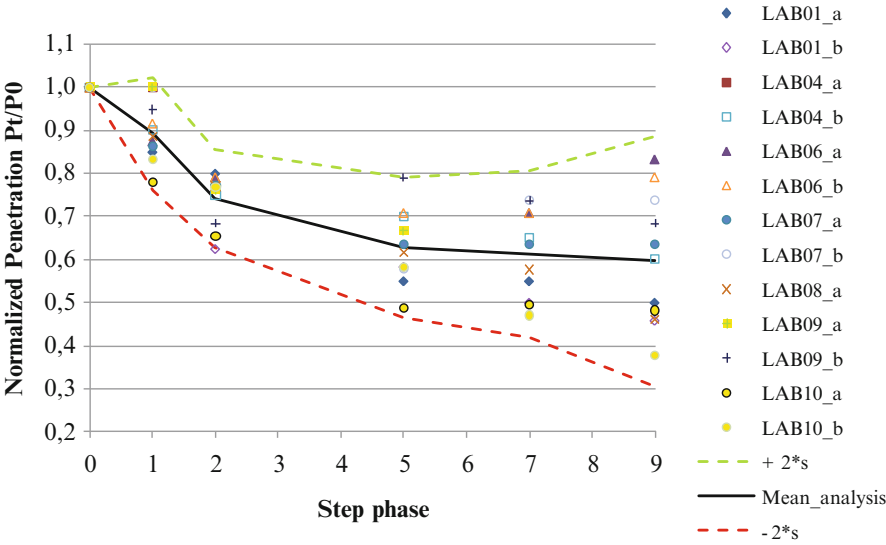


Fig. 7.9 Normalised penetration values for plant-mix

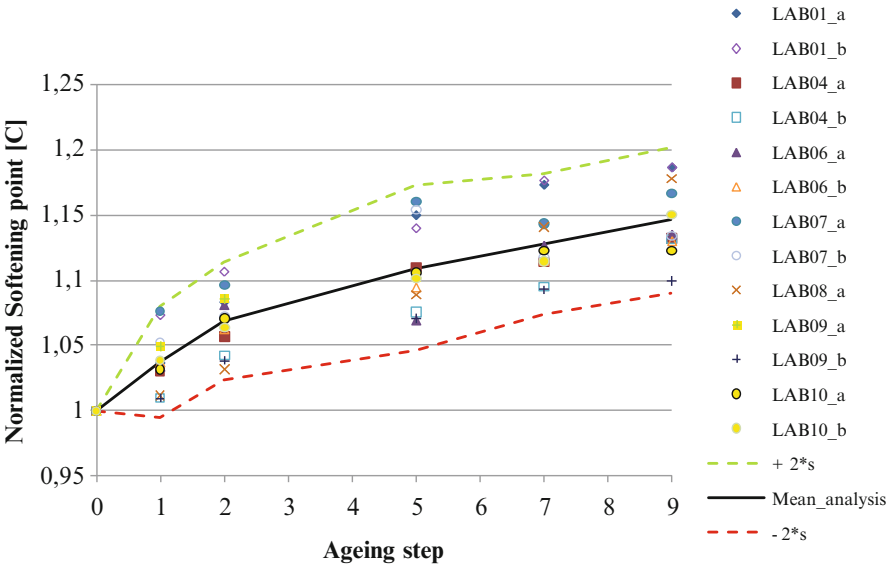


Fig. 7.10 Normalised results for RB-softening points of binder from plant-mix

After a first analysis by the rejections of outliers and a recalculation of the mean and the limits of the standard deviation, ‘new’ outliers appear. Since these outliers were not rejected in the first analysis, these results are retained.

In Fig. 7.11, the mean and standard deviation limits for the normalised penetration of the binders recovered from the laboratory and the plant-mixes are shown. The normalised values of Lab 5 (plant-mix without short-term ageing) are added.

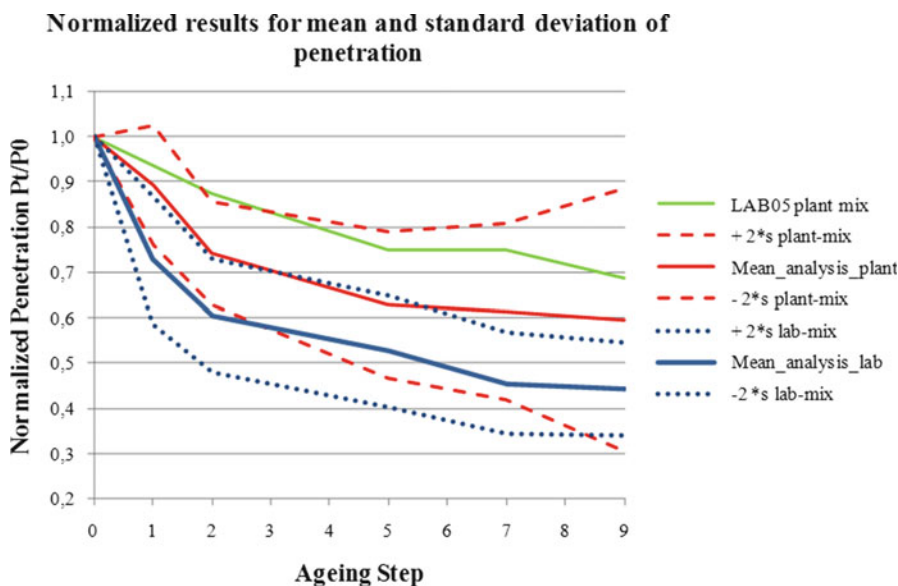


Fig. 7.11 Normalised results for mean and standard deviation of the penetration for lab-mix and plant-mix

The lab-mix shows a higher ageing effect during short-term ageing compared to the plant-mix. This could be due to two reasons: the initial ageing of the plant-mix during manufacture is more severe than in the laboratory and/or since the mixture is taken from the paver a part of the short-term ageing has already occurred. There is a parallel shift between the plant-mix and lab-mix during long-term ageing starting from ageing step 1.

From this analysis it is observed that the plant-mix aged without short-term ageing shows a nearly continuous decrease in penetration during long-term ageing. This inclination is also found by the mixtures during long-term ageing.

Similar results are demonstrated for the normalised RB-softening points in Fig. 7.12. The ageing process of the binders in the plant-mix is lower than for the lab-mix, due to the initial high RB-softening point of the plant-mix.

Also in each step, except for step 1, the interval of the mean ± 2 times s for the plant-mix is larger than for the lab-mix. This means that after the production step, the ageing process of the plant-mix is more random than the lab-mix.

From these results it is concluded that the plant-mix, batched in the paver, has already experienced ageing. This ageing is part of the short-term ageing process. The short-term ageing process in the round robin is apparently less intense if the mixture has already undergone an ageing process at high temperature.

The ageing process still continues after 9 days. After 5 and 7 days for the plant-mix and the lab-mix respectively, a decline of the ageing process is noted. At that point the absolute values of the results are similar as shown in Table 7.13.

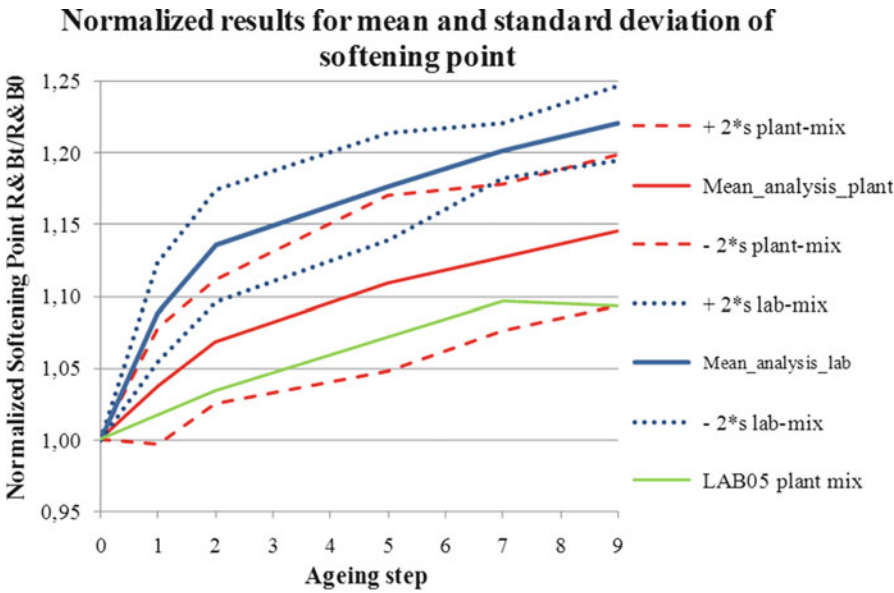


Fig. 7.12 Comparison of normalised results for mean and standard deviations of the RB-softening point for lab-mix and plant-mix

Table 7.13 Comparison of penetration and softening point data statistics for lab-mix and plant-mix during last step

			Penetration			RB-softening point		
			Mean	n	STDEV	Mean	n	STDEV
Mix type		Step	mm/10		mm/10	°C		°C
Lab-mix	After 7d LT	7	12.8	14	2.6	69.1	14	0.9
Lab-mix	After 9d LT	9	12.9	14	2.5	70.3	14	1.2
Plant-mix	After 5d LT	5	13.8	13	3.6	66.8	13	2.1
Plant-mix	After 7d LT	7	13.0	13	3.0	67.8	12	1.5

7.5.1.4 Repeatability of Individual Binder Extraction and Testing for the Laboratories

Since neither repeatability nor reproducibility values are given in literature for the extraction and recovery of binders after an ageing test, TG5 decided to evaluate the repeatability of the RRT by means of the delta (difference) of the penetration and RB-softening points results between sample a and sample b. In Figs. 7.13 and 7.14 respectively the calculated deltas of penetration and RB-softening point are given for lab-mix and the plant-mix.

The results of the penetration test and RB-softening point show no significant difference between samples a and b for the lab-mix and plant-mix. Note that in this analysis, the RB-softening points were reported as raw data (rounded to at 0.1

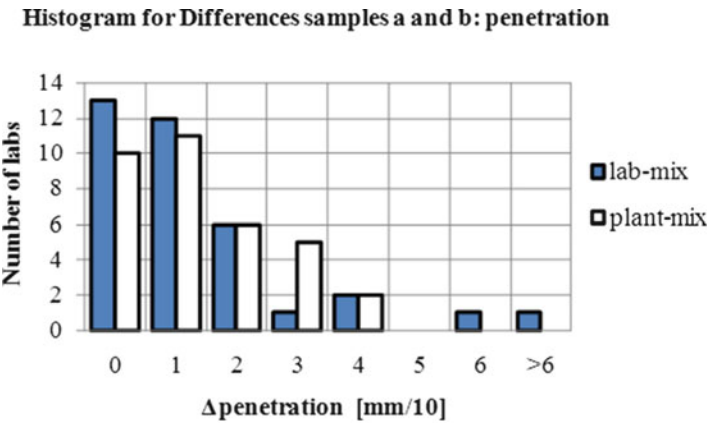


Fig. 7.13 Histogram for differences of penetration for sample a and sample b

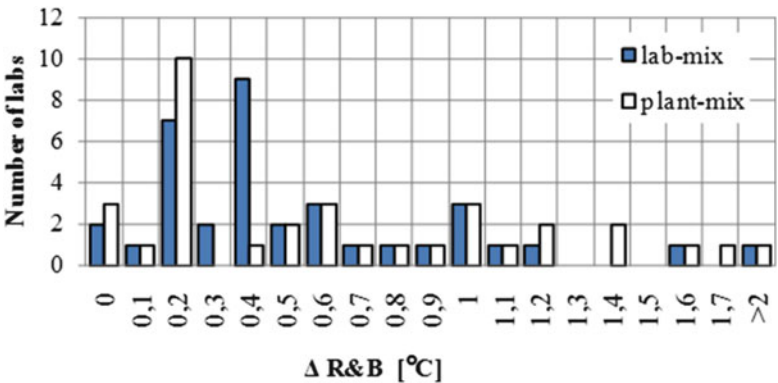


Fig. 7.14 Histogram for differences of RB-softening point for sample a and sample b

Table 7.14 Precision values EN12697-3

Test	Repeatability r	Reproducibility R
Penetration [1/10 mm]	0.1 * x	0.27 * x
Softening point (°C)	1.9	3.4

With x is the average of the results being compared

instead of 0.2) by most laboratories. This leads to delta 0.1 in the histogram for the RB-softening points. The differences shown in the figures can be compared to the r-value for the tests mentioned in the European Standard 12697-3 in Table 7.14.

In Table 7.15 a summary is given of the numbers of sample doublets (a and b) with their conformity to the r-value.

Table 7.15 Number of doublets related to r for penetration and RB-softening point results

	Penetration		RB-Softening point	
	Lab mix	Plant mix	Lab mix	Plant mix
Conform r-value	27	23	35	33
Non conform r-value	9	11	1	1

Table 7.16 R-value precision for the results of step 0 of plant-mix

	Penetration		Softening point	
Mean value	21.2	[1/10 mm]	60.2	°C
R-value (range)	5.7	[1/10 mm]	3.4	°C
Conform R-value	10		12	
Non conform R-value	3		1	

From this analysis it is clear that the repeatability for the RB-softening point is fulfilled. For the penetration however the repeatability is not fulfilled. The reason can be possibly attributed to the very low penetration values: most results have a penetration value around 20 [1/10 mm], meaning a r-value of 2 [1/10 mm] which is also the r-value for the penetration test on virgin binder (EN1426).

The reproducibility precision value can be calculated only from the data of plant-mix at step 0 (13 results). All other results of the plant-mix have been obtained from aged material which has been separately aged and are therefore not subjected to the R-value (see Table 7.16).

It can be concluded that the reproducibility of the extraction and recovery process by the different laboratories is acceptable. For the repeatability values the precision is difficult to meet, attributed to the very low penetration values and the ageing process.

7.5.1.5 Influence of Time Lap Between Ageing Process and the Extraction and Recovery Process

For the extraction and recovery process, two groups of laboratories are mentioned. Group 1 performed the binder recovery and binder tests following the chronological way of the ageing protocol (after manufacture, then after short-term ageing, then after 2 days with long-term ageing ...). Group 2 performed binder recovery and binder tests following the reverse order (first, after 9 days with long-term ageing, then after 7 days with long-term ageing, then 5 days with long-term ageing, ...). In the Figs. 7.15 and 7.16 the results for the mean penetration for lab-mix and plant-mix are shown.

It is observed that, in general, group 1 obtains higher values for penetration than group 2. For the results of the RB-softening point, group 1 find lower results for the lab-mix. For the plant-mix the differences are too small to draw conclusions. From these few results, it can be concluded that the differences are small but that group 2 shows the hardest binder after recovery.

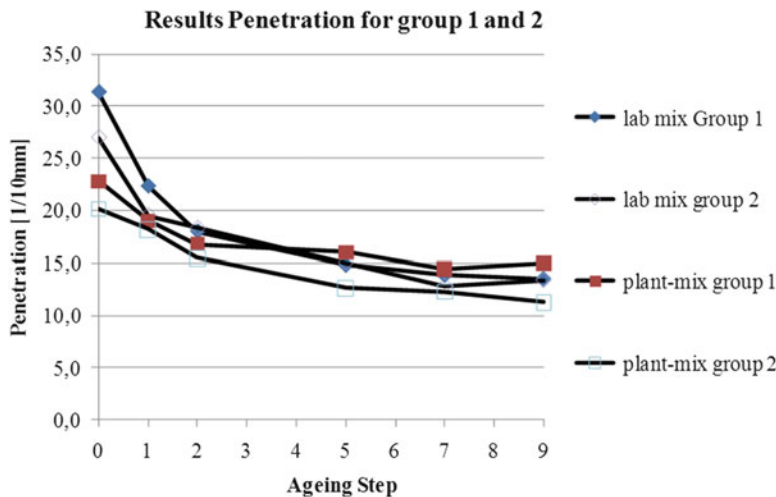


Fig. 7.15 Results of means of penetration for group 1 and group 2 (different way of extraction and recovery)

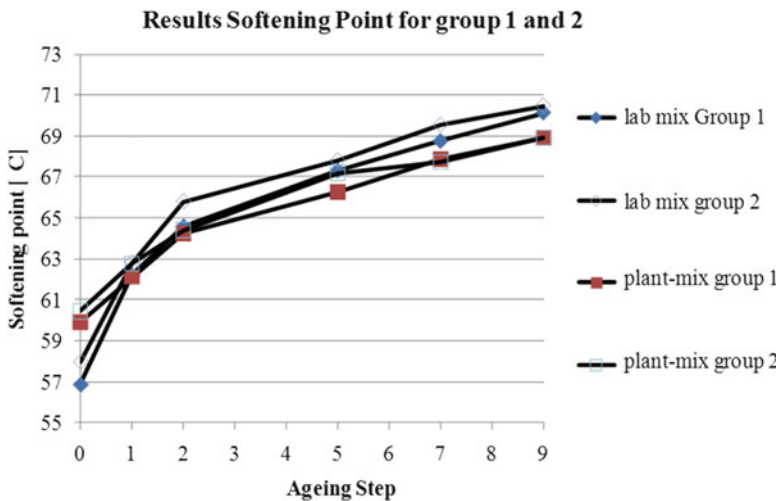


Fig. 7.16 Results of mean of the RB-softening points for group 1 and group 2 (different way of extraction and recovery)

7.5.1.6 Conclusions of the Penetration and RB-Softening Point Study

In general, it can be concluded that the results are quite consistent taking into account the mixing procedure, ageing procedure, extraction/recovery and test methods. The shifting is probably due to differences in methods.

To show the trends, the data are expressed in relative values normalised to 1, for step 0 corresponding to sampling after manufacturing. Based on the mean value of

Table 7.17 Results of ageing tests on virgin binders: Penetration and softening point

	Lab 3		Lab 6		Lab 7	
	Pen (1/10 mm)	RB (°C)	Pen (1/10 mm)	RB (°C)	Pen (1/10 mm)	RB (°C)
Virgin binder	37	54.1	35	54.1	36	54.2
RTFOT	26	58.6	25	59.7	25	60.2
RTFOT + PAV	20	66.8			17	64.2

penetration and RB-softening point it can be concluded that asphalt mixtures manufactured in the laboratory and heated in an air ventilated oven age resulting in a harder binder. This hardening of the binder can be described as a decrease in penetration. After 9 days the binder still hardens although the rate of this ageing decreases. A penetration more than 50% of its initial value (after production) was obtained after 7 days.

For the plant-mix, the ageing rate during long-term ageing is similar to the lab-mix. Since this mixture is sampled in the paver, a part of the short-term ageing has already occurred. This aspect can be noticed by the lower rate of ageing during short-term ageing and the shift of the results during long-term ageing. After 7 days of ageing for both mixtures, the difference in short-term ageing is no longer noticeable.

One laboratory has aged the asphalt mixture without short-term ageing. After 7 days long-term ageing the results are similar to the results of the mixtures with short-term ageing. However, the initial penetration is very low compared to other laboratories. More research of the influence on the short-term ageing period on the total ageing process is recommended.

From the viewpoint of penetration and RB-softening point, a short-term ageing of 4 h at 135 °C and a long-term ageing of 7 days at 85 °C will give reproducible results for ageing an asphalt mixture in an air ventilated oven.

7.5.1.7 Ageing Tests on the Reference Binders

Three laboratories have tested the penetration and RB-softening point of the virgin binder and after short-term and long-term ageing tests for binders: RTFOT, PAV and RCAT. In Table 7.17, the results are shown for RTFOT and RTFOT + PAV.

One laboratory has performed tests with the RCAT on the virgin binder. In Table 7.18, the results are given. Samples are taken at each step, similar to the batch ageing. Penetration is calculated by using the Saal and Labout equation.

The results of the RTFOT and RCAT short-term binder ageing procedures are compared to the characteristics of the recovered binder from the laboratory and plant-mixtures for step 0 (after production) and step 1 (after short-term ageing).

From the results, shown in Figs. 7.17 and 7.18, it is concluded that short-term binder ageing is more similar to the ageing of the binder during production of the mixture. The ageing of the binder in the mixtures after short-term ageing (step 1) is higher than for the binder ageing. The minimum and maximum values of the results for the lab-mixture and plant-mix ageing are given.

Table 7.18 Results of RCAT ageing

Lab 6		Penetration G*_8mm	RB-Softening point
		1/10 mm	°C
Virgin	−1	36.7	54.1
RCAT short-term	1	28	58.5
RCAT LT 2d	2	20	64.3
RCAT LT 5d	5	16	68.9
RCAT LT 7d	7	14	69.6
RCAT LT 9d	9	13.5	72

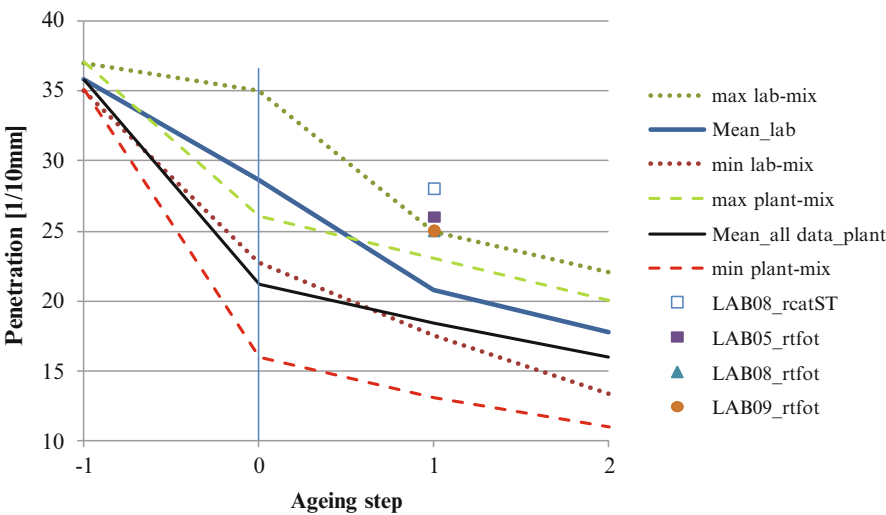


Fig. 7.17 Comparison binder ageing RTFOT and RCAT ST with asphalt mixture ageing – penetration (without rejection of outliers)

In Fig. 7.19, the results of the long-term ageing tests on the binder are shown together with all data of the results of the mixture ageing.

It is clear that the RCAT long-term ageing after 2 and 5 days is similar to ageing by PAV and that the penetration and RB-softening point results of RCAT are similar to ageing in the asphalt mixtures (Fig. 7.20).

7.5.1.8 Penetration Value vs. RB-Softening Point

As ageing affects both penetration and RB-softening point, it is interesting to analyse the correlation of these both parameters. This correlation consists in displaying penetration values on a log scale vs. RB-softening point values. This allows to compare with pen-grade specification boxes and also to analyse the evolution as it may be done with production distillation curves. A full in-depth

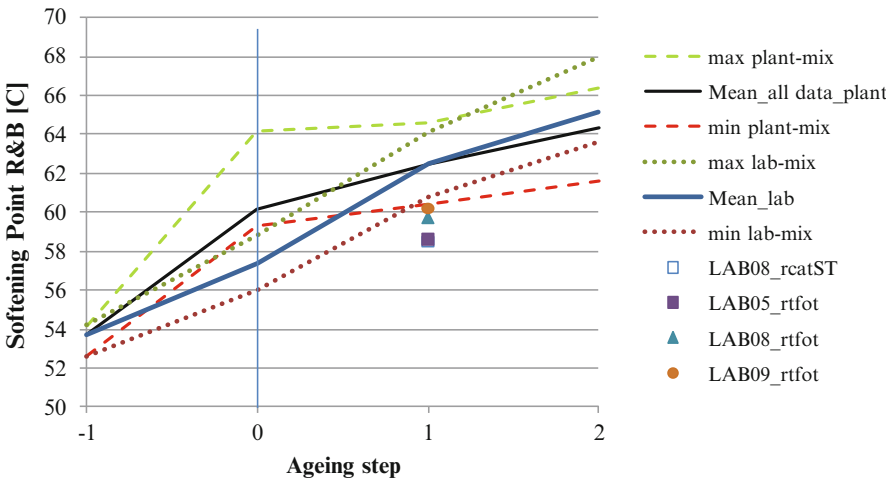


Fig. 7.18 Comparison binder ageing RTFOT and RCAT ST with asphalt mixture ageing – RB-softening point (without rejection of outliers)

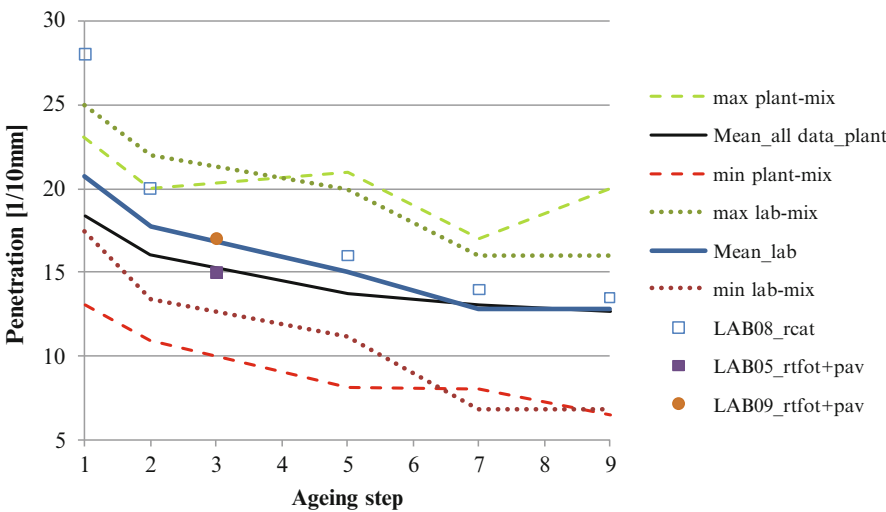


Fig. 7.19 Comparison binder ageing RTFOT + PAV and RCAT LT with asphalt mixture ageing – penetration

analysis can provide a lot of comprehensive information but may be outside the scope of the RRT. Thus the analysis hereby is restricted to the dispersion of the results provided by the laboratories and comparison between lab-mixes and plant-mixes.

In this analysis, penetration values are displayed on a log-scale that enlarge the small variation at the bottom of the scale (above 10) compared to higher values at

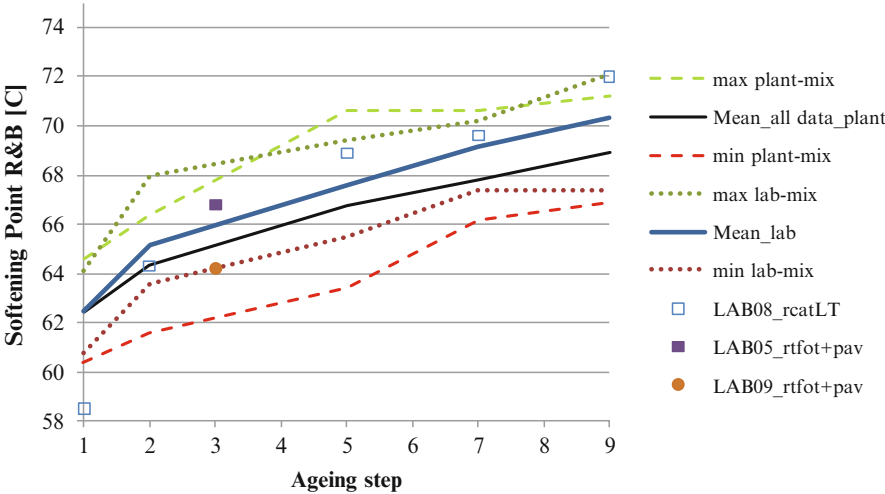


Fig. 7.20 Comparison binder ageing RTFOT + PAV and RCAT LT with asphalt mixture ageing – RB-softening point

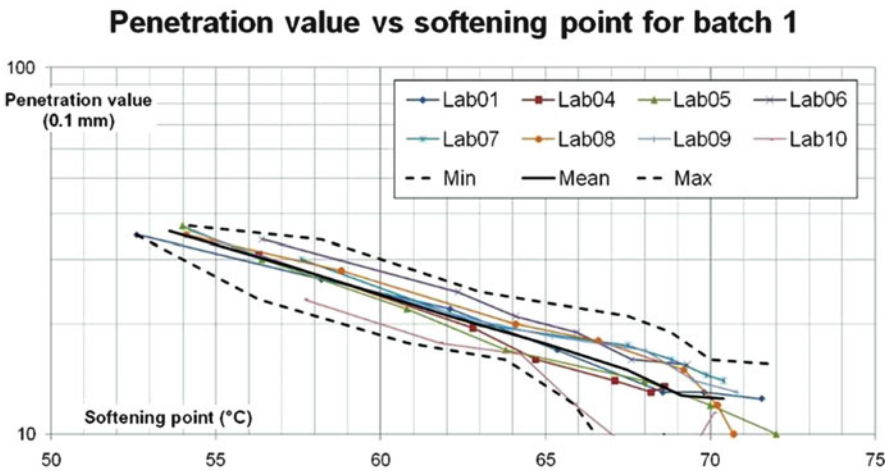


Fig. 7.21 Penetration values vs. RB-softening point values for the lab-mixes

the top of the scale (above 20). The RB-softening point is displayed on a linear scale. Usually ageing is shifting to the bottom right of the graph, the further to the right the more oxidation has occurred.

For lab-mix 1, the results from all laboratories are displayed in Fig. 7.21. From this figure, it is confirmed that results of lab 10 are on the bottom edge of the whole set of data and affect the mean value calculation while the rest of the data is within a narrow range. In the later analysis, the data from lab 10 is rejected. Furthermore, in the graph below only the mean values are displayed, with some error bars to encompass the minimum and maximum values of each couple

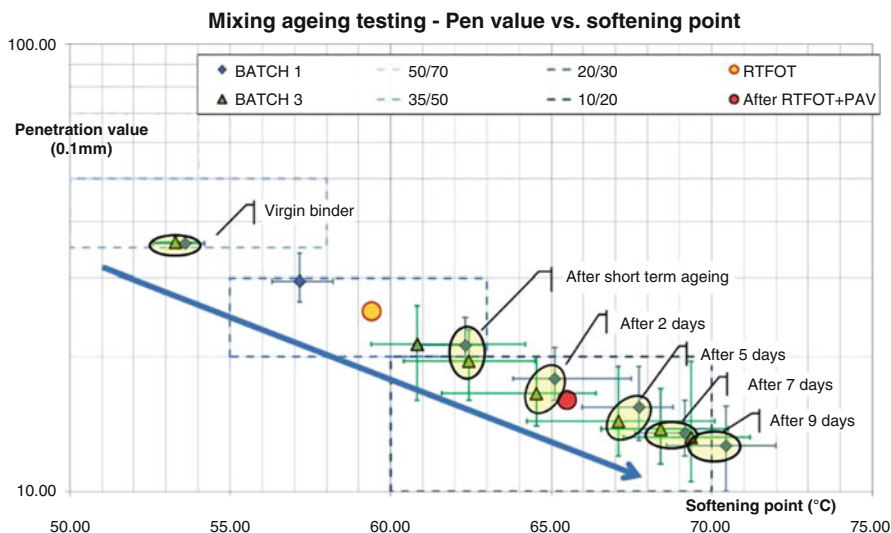


Fig. 7.22 Mixing ageing testing – Pen value vs. RB-softening point

of values for laboratory and plant-mixes. For a comparison, the results after RTFOT and PAV + RTFOT are displayed. The analysis of this graph shows:

1. Interestingly, it is worth noting that there is a limited correlation just after the mixing process between lab-mix and plant-mix as well as RTFOT on the binder. The plant-mixing process is more severe than the lab-mixing process. The main difference between the two processes is that in laboratory, aggregates are kept at temperature for a whole night, leading to a homogenous temperature. While at plant, aggregates are heated up in a short time meaning that the outside of stones are certainly much higher than the inside and the mean temperature.
2. This difference after mixing is however, rapidly offset as the both values after short-term ageing are close together, at least in the range of the error bars.
3. Afterwards, there is a shift in the difference between laboratory and plant-mixes. After the short-term ageing, the RB-softening point results are similar; the difference is more on the penetration value. After 9 days, the penetration values are almost equal while the difference is more for the RB-softening point results. However, altogether all data are consistent within the error bars between laboratory and plant-mixes (Fig. 7.22).

Some trends may be found. Figure 7.23 displays some regression curves for both batches. In this semi-log scale it displays a straight line. The regression is not bad with a R-value of $R^2 = 0.99$. The line follows pretty much the pen-grade specification boxes. After short-term ageing the binder has lost one pen-grade (from 35/50 to 20/30) and after long-term ageing another, (from 20/30 to 10/20). This is usually the trend that is encountered on site. It is worth to compare with blowing bitumen, as a result of extra oxidation. For blown bitumen the boxes will go further to the right, meaning that for a given penetration value, the RB-softening

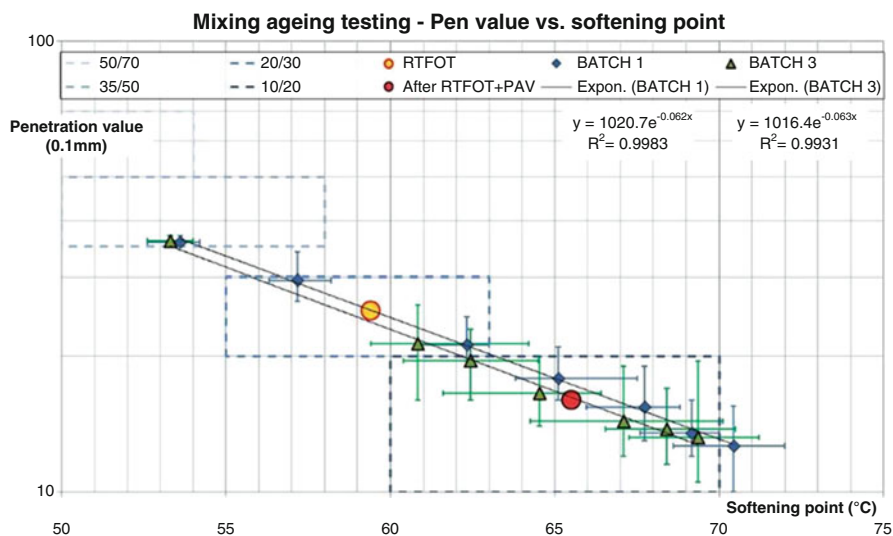


Fig. 7.23 Mixing ageing testing – Pen value vs. RB-softening point

point is greater (boosting RB-softening point). Maybe it occurs at the end of the test, after 7 days, where the penetration value does not decrease anymore while the RB-softening point continues to increase as a result of extra oxidation.

In conclusion, differences do exist after the mixing process between laboratory and plant. After long-term ageing the mixing process has a limited effect on the final results.

7.5.1.9 Interpretation Through the Penetration Index

In addition from the analysis of penetration value and RB-softening point, another interpretation can be made according to the penetration index (PI). PI is calculated with the following formula, where Pen is the penetration at 25 °C and TR&B is the Ring & Ball temperature (RB-softening point):

$$PI = \frac{20 - \frac{500(\log(800) - \log(Pen))}{TR\&B - 25}}{\frac{500(\log(800) - \log(Pen))}{TR\&B - 25} + 1} \quad (7.1)$$

Figure 7.24 displays the evolution of the calculated penetration index over the test duration for both the laboratory and plant-mixes and is compared to the RTFOT and PAV after RTFOT (arbitrarily set after 5 days) values. Only mean values are plotted with error bars for minimum and maximum values. Here, the values from lab 10 were not taken into account.

The trend is an increase of the penetration index from about –1 to 0. This is in line with common observation from field behaviour. However it seems that this

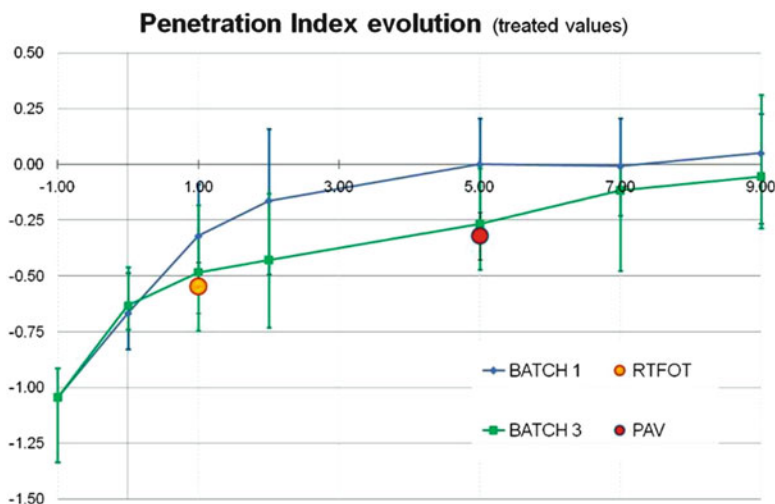


Fig. 7.24 Penetration index evolution

trend is faster for the mix made in laboratory while this difference tends to be offset at the end. Also, there is a tendency to reach a plateau after 5 days when considering the error bars.

7.5.2 DSR Results on Recovered Bitumen

Dynamic shear rheometers (DSRs) were used to assess the rheology of the recovered binder. The first parameter looked at to compare binder data from different conditions was G^* at 25 °C and 0.4 Hz. As well as looking at the recovered binder from the mixing and ageing process, the virgin binder, virgin binder aged using the Rolling Thin Film Oven Test (RTFOT), virgin binder aged using the Rolling Thin Film Oven Test (RTFOT) followed by 20 h at 100 °C in a Pressure Ageing Vessel (PAV) and virgin binder aged for 25 h at 100 °C in a PAV were also looked at. These were used to assess the ageing that had been undertaken on the mixtures.

Looking at the G^* values in Fig. 7.25 it can be seen that the virgin binder has a complex modulus of 7.5×10^5 Pa. After RTFOT ageing, this has increased to 1.5×10^6 Pa. After further ageing with the PAV, the G^* has increased to approximately 3.3×10^6 Pa. The use of PAV at 100 °C for 25 h was also looked at as this was supposed to be equivalent to RTFOT + PAV at 100 °C for 20 h. After PAV for 25 h the G^* had increased to 3.4×10^6 Pa, which is very similar to RTFOT + PAV ageing as expected. Figure 7.26 shows the effect of ageing on phase angle. This follows the reverse trend to the complex modulus, with the phase angle δ reducing from 68° in the virgin binder to 62° after RTFOT. After RTFOT + PAV it has decreased to 53°, which is similar to the phase angle after PAV for 25 h, which is 55°.

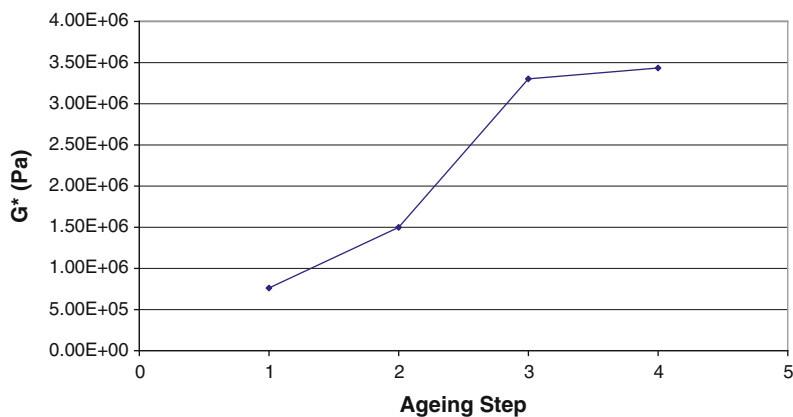


Fig. 7.25 The effect of ageing of virgin binder on G^* at 0.4 Hz at 25 °C

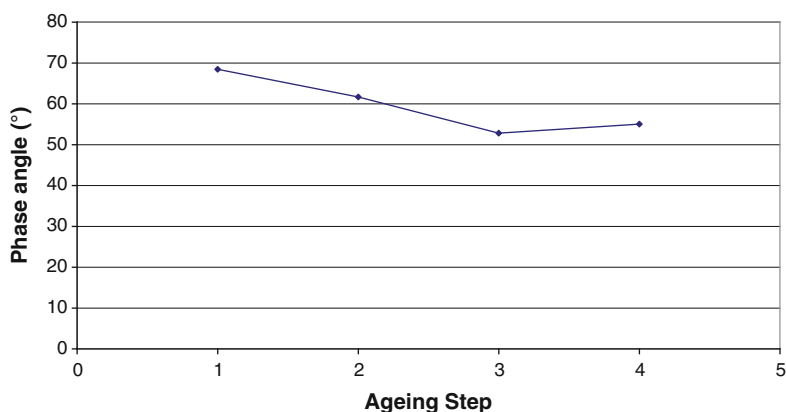


Fig. 7.26 The effect of ageing of virgin binder on phase angle at 0.4 Hz at 25 °C

Figure 7.27 shows the effect of ageing step on the G^* of Lab-mix 1. Again it can be seen that G^* increases with ageing step from approximately 1.0×10^6 after mixing to 2.0×10^6 after short-term ageing. It then increases to 2.9×10^6 , 3.6×10^6 , 4.4×10^6 and 4.6×10^6 after 2, 5, 7 and 9 days respectively. Figure 7.28 shows the effect of ageing step. This again follows a similar trend to the ageing of the virgin binder, with phase angle δ decreasing with ageing step. The phase angle is 64° after mixing and drops to 57° after short-term ageing. It then decreases to 54, 51, 50 and 49 after 2, 5, 7 and 9 days respectively. Figure 7.29 shows the effect of ageing step on the G^* of plant-mix. Again it can be seen that the G^* increases with ageing step from approximately 1.8×10^6 after mixing to 2.7×10^6 after short-term ageing. It then increases to 3.2×10^6 , 4.2×10^6 , 5.1×10^6 and 5.5×10^6 after 2, 5, 7 and 9 days respectively. Figure 7.30 shows the effect of ageing step. This again follows a similar trend to the ageing of the

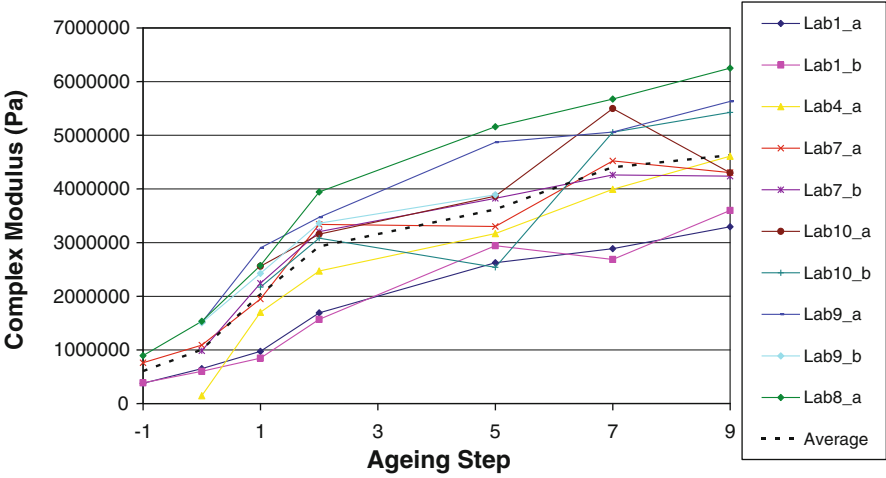


Fig. 7.27 The effect of ageing step on the G^* of lab-mix at 0.4 Hz at 25 °C

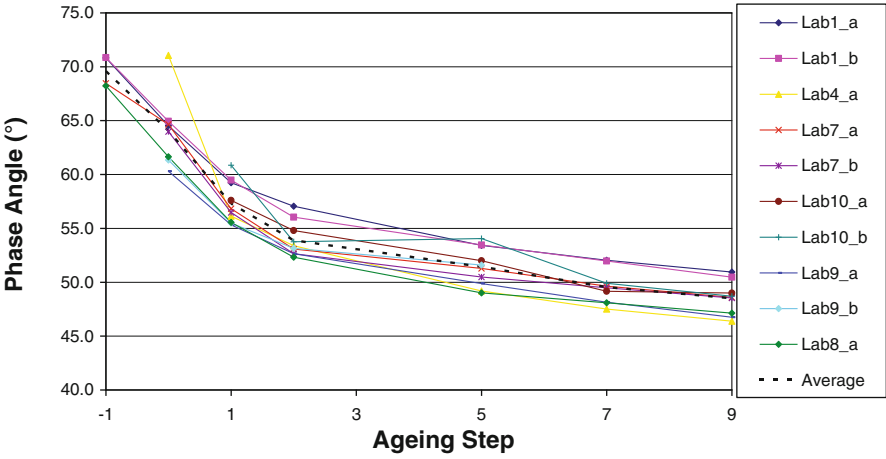


Fig. 7.28 The effect of ageing step on δ of lab-mix at 0.4 Hz at 25 °C

virgin binder, with phase angle δ decreasing with ageing step. The phase angle is 59° after mixing and drops to 55° after short-term ageing. It then decreases to 53, 49, 48 and 46 after 2, 5, 7 and 9 days respectively.

Looking at the comparison of the average G^* values and average phase angle values for the two batches (Figs. 7.31 and 7.32) it can be seen that the lab-mix is roughly one time step behind the plant-mix in terms of its ageing. The G^* values after mixing for the plant-mix are approximately the same as the lab-mix after short-term ageing and after short-term ageing the plant-mix values are approximately the same as the lab-mix after 2 days ageing etc.

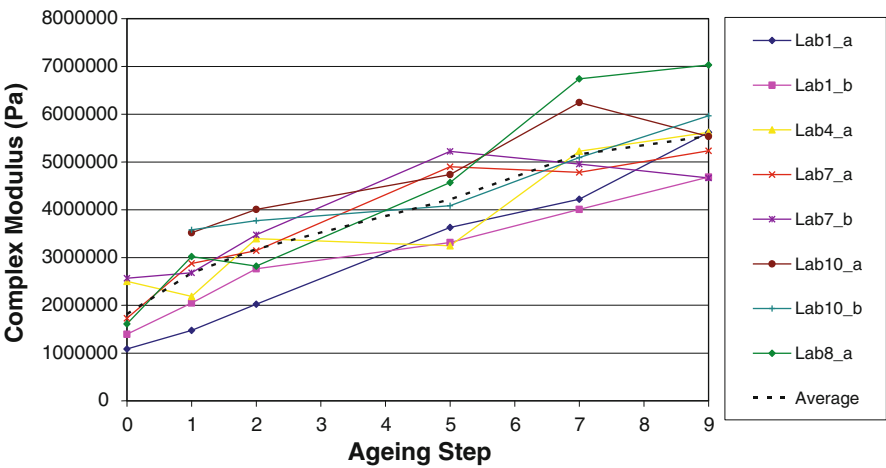


Fig. 7.29 The effect of ageing step on the G^* of plant-mix at 0.4 Hz at 25 °C

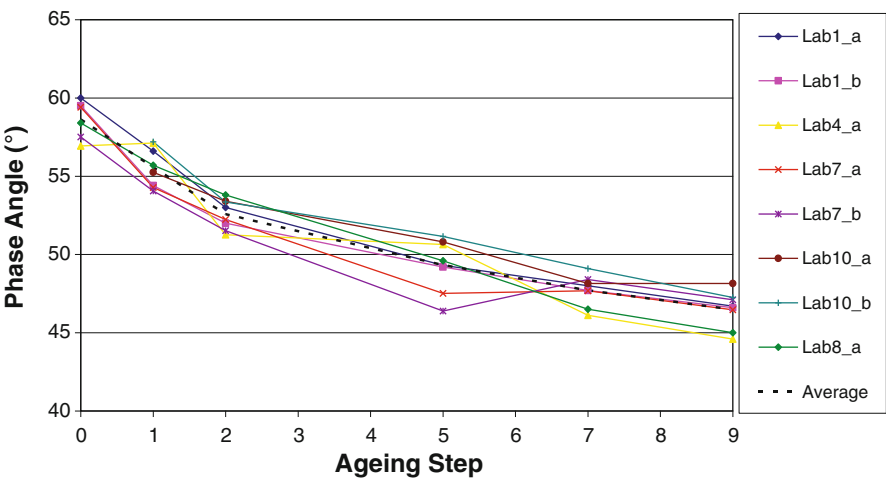


Fig. 7.30 The effect of ageing step on δ of plant-mix at 0.4 Hz at 25 °C

Frequency sweeps were carried out from 0.1 to 10 Hz at 11 different frequencies using DSRs over a range of temperatures (−5 to 55 °C at 10 °C intervals). These were carried out in order to be able to construct master curves of both complex modulus and phase angle at specific reference temperatures. Master curves were then constructed using 25 °C as the reference temperature. From Fig. 7.33, showing the complex modulus master curves, it can be seen as the virgin binder is aged using RTFOT, RTFOT + PAV 20 h and PAV 25 h, the bitumen gets stiffer. The Master curves for RTFOT + 20 h PAV and 25 h PAV are very similar indicating that these ageing processes have a similar effect on the properties of the binder. Figure 7.34,

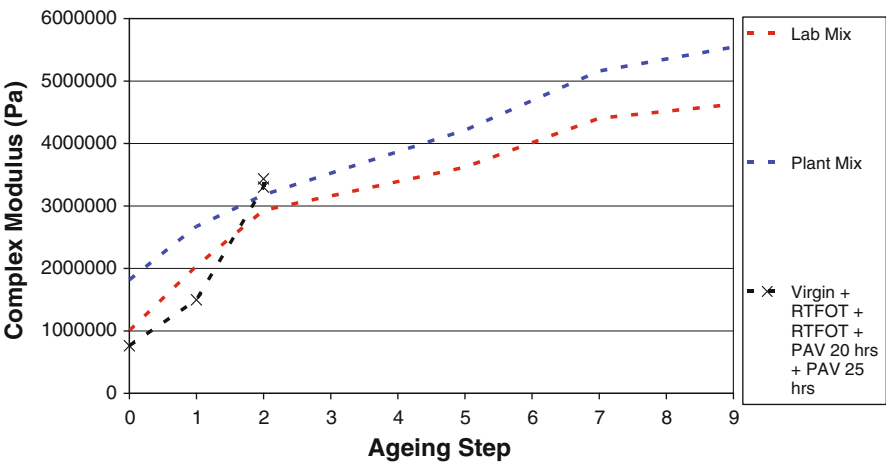


Fig. 7.31 The effect of ageing step on the G^* of lab-mix and plant-mix at 0.4 Hz at 25 °C

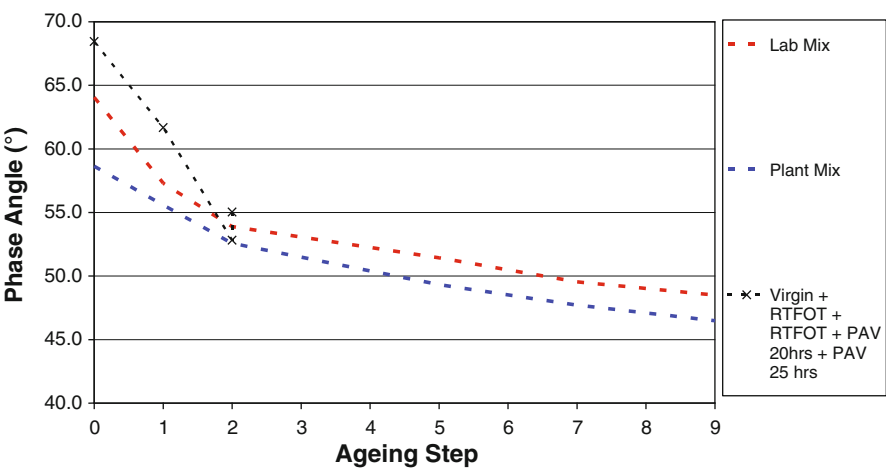


Fig. 7.32 The effect of ageing step on δ of Batches 1 and 3 at 0.4 Hz at 25 °C

showing the phase angle master curves, it can be seen as the virgin binder is aged using RTFOT, RTFOT + PAV 20 h and PAV 25 h, the phase angle is reduced. Again the Master curves for RTFOT + 20 and 25 h PAV are very similar indicating that these ageing processes have a similar effect on the properties of the binder.

Figure 7.35 shows the effect of ageing step on the complex modulus master curves of the ageing steps applied to lab-mix. Again it can be seen that the G^* increases with ageing step. Figure 7.36 presents the effect of ageing step on the phase angle master curve. This again follows a similar trend to the ageing of the virgin binder, with phase angle δ decreasing with ageing step. Figure 7.37 shows the effect of ageing step on the complex modulus master curve of plant-mix. Again

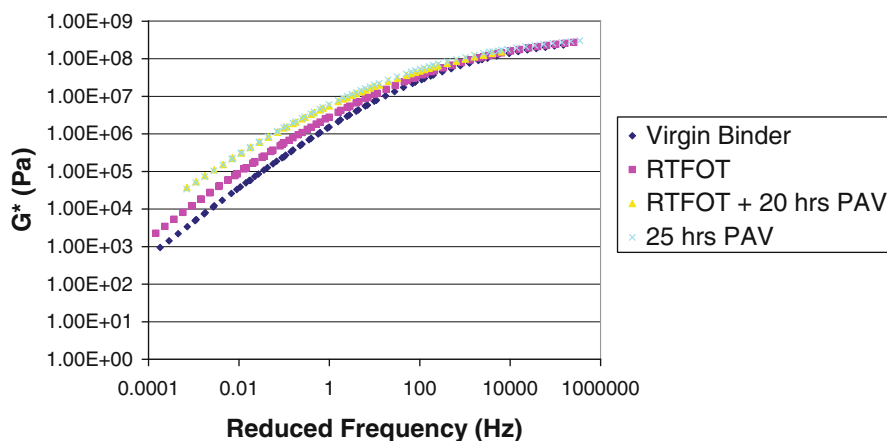


Fig. 7.33 The effect of ageing step on the G^* master curve at 25 °C of the virgin binder and aged virgin binder

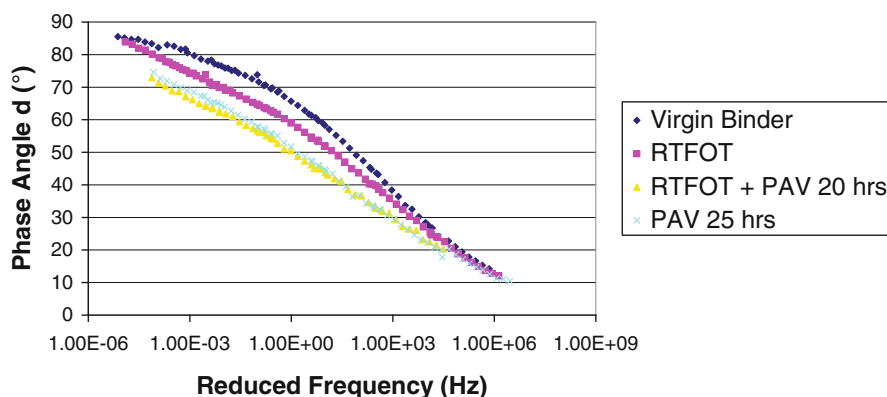


Fig. 7.34 The effect of ageing step on the phase angle master curve at 25 °C of the virgin binder and aged virgin binder

it can be seen that the G^* increases with ageing. Figure 7.38 depicts the effect of ageing step on the phase angle master curve. This again follows a similar trend to the ageing of the virgin binder and lab-mix, with phase angle δ decreasing with ageing step.

The rheological study shows the overall evolution of the ageing protocol. In both the laboratory and the plant-mixes, the overall trend is for an increasing complex modulus G^* with ageing step. There is quite a bit of scatter in the G^* results, but this is to be expected as the testing was carried out in several laboratories and therefore local procedures can have an effect on the results. However, the general trends for all the laboratories are the same. The phase angle δ tends to decrease with increasing ageing step as expected for a binder that is being aged. Figs. 7.35, 7.36, 7.37 and 7.38 show some typical master curves for both the lab-mix and the plant-mix aged using the protocol and then tested after different time steps.

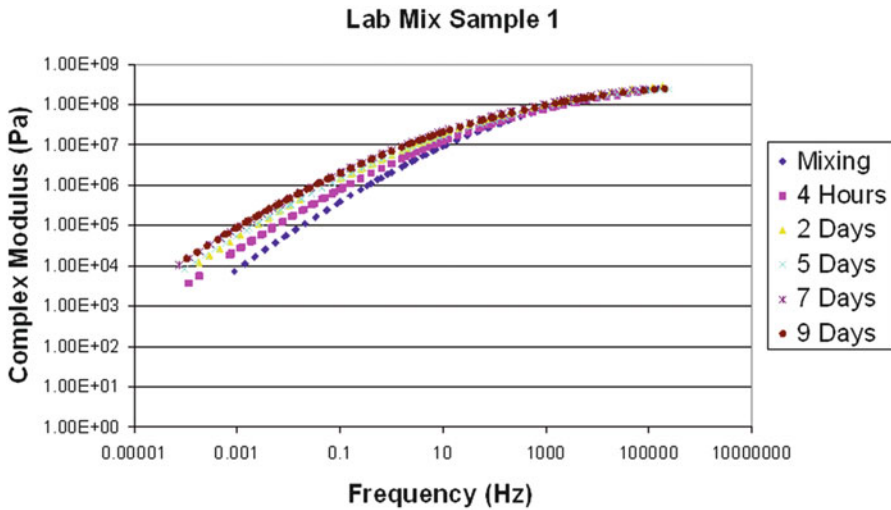


Fig. 7.35 The effect of ageing step on the G^* master curve of lab-mix

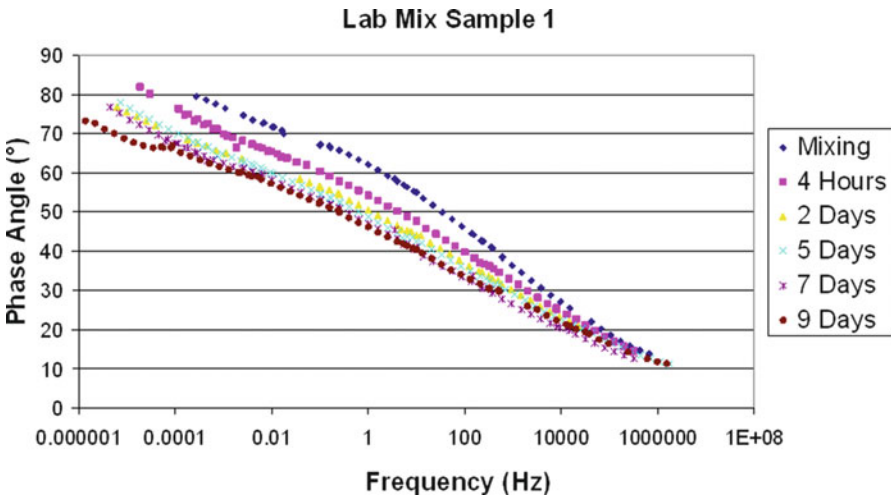


Fig. 7.36 The effect of ageing step on the phase angle master curve of lab-mix

Again, in both mixtures the general trend is for G^* to increase with increasing ageing step and for the phase angle δ to decrease. Figures 7.31 and 7.32 show the average values for both complex modulus G^* and phase angle δ for both the laboratory and the plant-mixes aged according to the protocol. These figures also show the G^* and δ values associated with typical short-term RTFOT ageing and PAV long-term ageing. These figures show that the RTFOT ageing is roughly equivalent to the short-term ageing used in the protocol, whereas the PAV ageing only appears to be equivalent to approximately 3–4 days of the long-term ageing protocol used in the RTT.

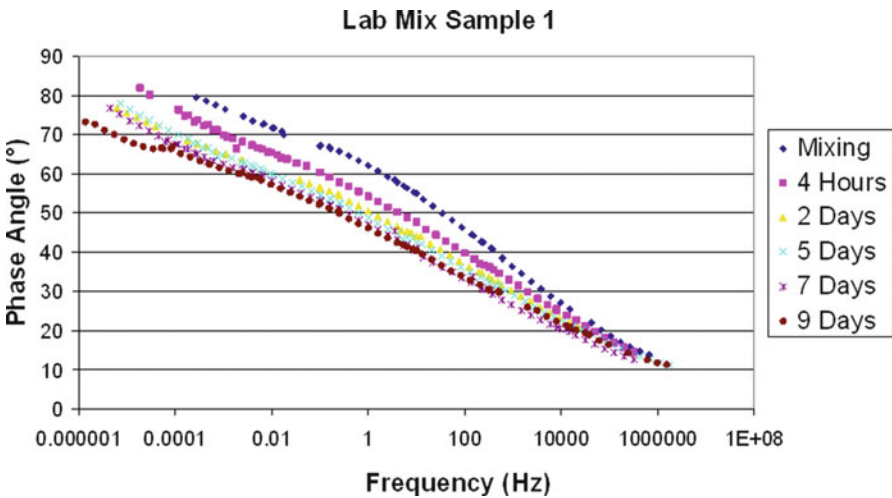


Fig. 7.37 The effect of ageing step on the G^* master curve of plant-mix

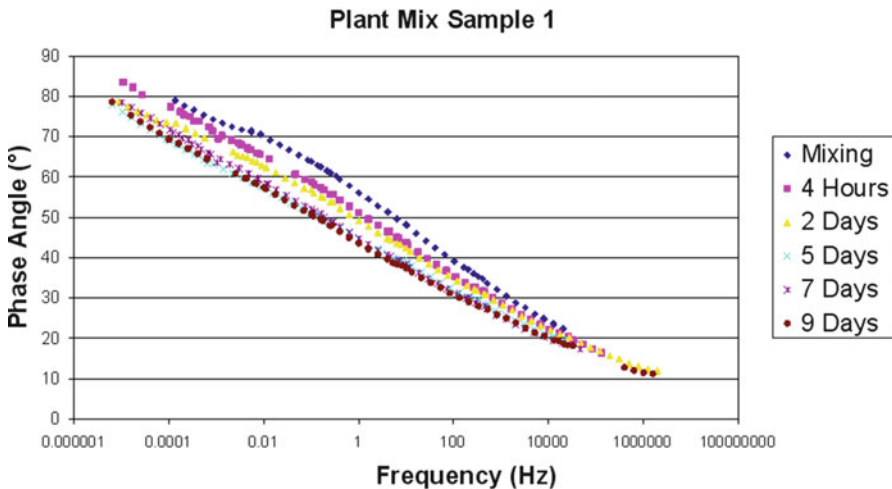


Fig. 7.38 The effect of ageing step on the phase angle master curve of plant-mix

7.5.3 Oxidation Results Using Fourier Transform Infrared (FTIR)

7.5.3.1 Results of the Round Robin Test

Eight participants have performed FTIR tests on laboratory and plant-mixes. Oxidation indices are obtained following the LCPC method n°69 (Mouillet et al. [16]). The principle of FTIR measurement and method 69 are detailed in Annex: Principle of infrared spectroscopy. In the following study, the limits of integration

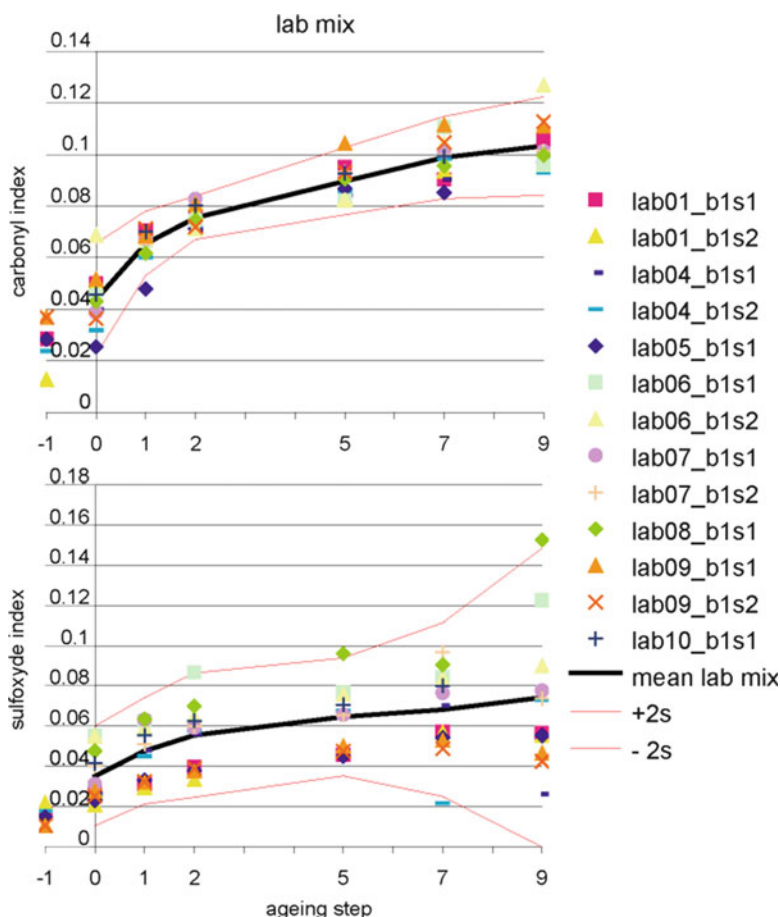


Fig. 7.39 Comparison of the ageing of the samples of the lab-mixes of all the laboratory. Carbonyl and sulfoxide indices are both presented

for calculating these areas have been defined for the analysis of all the tests. These limits of integration (wavenumbers) are the following: the Carbonyl area is de-fined between $1,660$ and $1,753\text{ cm}^{-1}$, the sulfoxide area is defined between $1,047$ and 995 cm^{-1} , and the base area is defined between $1,525$ and $1,350\text{ cm}^{-1}$.

7.5.3.2 Oxidation of the Lab-Mix During the Ageing Procedure

Results, in terms of oxidation indices, of the RTT performed on the lab-mix are presented in Fig. 7.39. The x-axis represents the ageing step, with the steps $[-1, 0, 1, 2, 5, 7, 9]$ representing the original binder, the age step after mixing, the step just after the short-term procedure and after 2, 5, 7 and 9 days of the long-term ageing procedure respectively. The carbonyl and the sulfoxide indices are both presented.

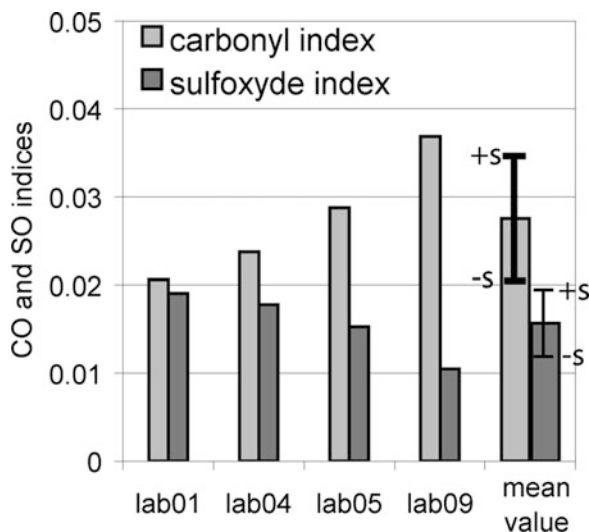
According to the general evolution of Ico and Iso for all the laboratories during the ageing procedure, it can be concluded that these oxidation indices may be good predictors for the determination of oxidation level. The Ico shows a more significant evolution than the Iso. This may either mean that the Ico is more efficient, or these two indices show the differences between the kinetics of oxidation of sulfoxide and carbonyl molecules. The two indices increase along the ageing steps, but they seem to evolve less and less. Between steps 2 and 5, the evolution is more or less constant, with a slight decrease between steps 7 and 9. The evolution is more significant between the steps 0 and 1. It is not surprising to observe that the short-term ageing has a stronger influence on ageing than the long-term ageing, due to a higher temperature. Considering the total evolution, the short-term ageing has approximately the same impact on the evolution of the indices than the whole long-term procedure. It has to be noticed that on a real time scale, the short-term ageing only represents 0.17 days (4 h), and not even 1 day. Thus, the slope between step 0 and 1 would be definitely more significant.

The repeatability and the reproducibility of this RTT have been assessed. An analysis of the results laboratory by laboratory highlights a good repeatability for each laboratory, on the ageing of the lab-mix. However, the comparison of the results of all the laboratories shows more scattering and then less reproducibility. It can also be noticed that the Iso presents less relative evolution and more scattering than the Ico one. In general, the trend consists of an increase of the two indices for each laboratory. Iso results are less reproducible than Ico results. Fillers in the recovered bitumen may explain the more significant scattering, because of its influence on the calculation of the Iso. The presence of filler inside the bitumen may interfere with the calculation of Iso, because sulfoxide and filler regions are very close in spectra. A second explanation may be that there is less sulfoxide creation than carbonyl creation.

It has to be noted here that the scattering between results of the different laboratories not only involves differences in the way to follow the ageing protocol, but it can also include differences in the way to manufacture the loose mix and the way to extract and recover the binder. It is therefore meaningful to observe more repeatability than reproducibility.

Oxidation indices have been determined by several laboratories on the original binder. Results are presented in Fig. 7.40. Whatever the index, Ico or Iso, an important scattering can be observed between the results. Theoretically, we would expect that these results would be the same. Two reasons may lead to the observed differences. The first may be that after being quartered, the binder has not been aged the same way in the respective boxes. The second is due to differences between the ways that binders are extracted and recovered, which may lead to different oxidation levels despite a same initial state. It has also been envisaged that the time of quartering may be a possible cause explaining the differences. At this time, determining the exact cause remains difficult. However, the scattering observed in the results during the ageing protocol are not less than the scattering of the results of the original binder.

Fig. 7.40 Results of FTIR tests performed on the original binder



7.5.3.3 Oxidation of the Plant-Mix During the Ageing Procedure

In Fig. 7.41, the evolutions of oxidation indices Ico and Iso are presented as a function of the ageing step.

The same trends as for the lab-mix can be observed. An evolution of the both indices is highlighted by the mean value. The ageing evolution, in terms of slope, is more important during the short-term ageing than during the long-term ageing (Fig. 7.41 may cause the reader an error due to the fact that the short-term ageing, which only represents 4 h of ageing, is at step 1). Ico evolves more relatively than Iso and the scattering is more significant for Iso than for Ico. In this study, the very high Iso result obtained by lab 08 at step 1 has been correlated to the presence of filler in the extracted bitumen, which pollutes the measurement of the Iso.

The origin of the scattering observed during this RTT may be due to several reasons. Contrarily to the RTT performed on the lab-mix, the scattering cannot be related to differences in manufacturing the material because all the samples come from the same plant-mix. If the samples are different, this can only be due to heterogeneity in the material and a problem with the sampling. Differences between the processes of extraction and recovery may also be responsible for the scattering.

7.5.3.4 Comparison of the Evolutions of Ageing Between Plant- and Lab-Mixes

A comparison of the evolution of ageing between the laboratory and plant-mixes and was performed. The results are presented in Fig. 7.42. The comparison first shows that the lab-mix and the plant-mix have different evolutions. The lab-mix has a higher global Ico and a lower global Iso than the plant mix. These two results seem to be contradictory based on the initial oxidation level. Thus no information

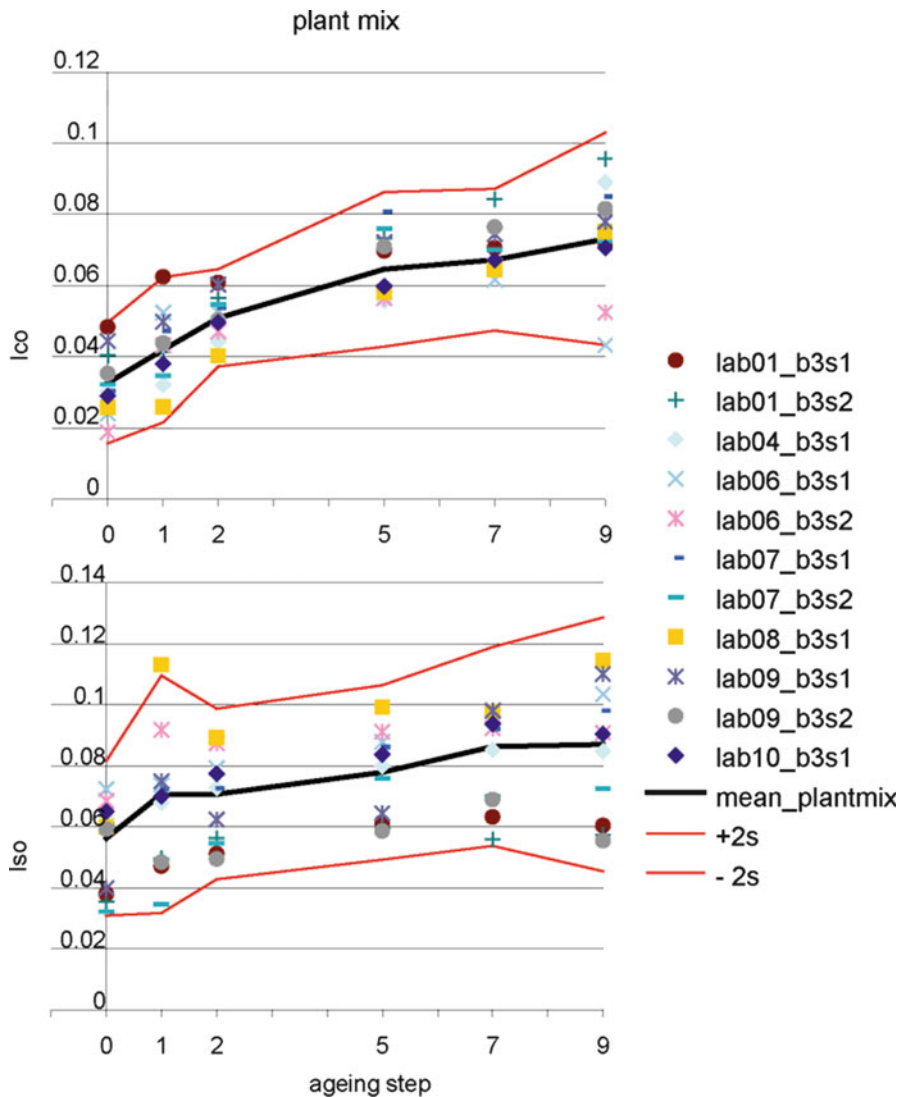


Fig. 7.41 Comparison of the ageing of the samples of the plant-mixes of all the laboratories. Carbonyl and sulfoxide indices are both presented

can really be found from these absolute values. Relatively, the lab-mix evolves more than the plant-mix, in the case of both the Ico and Iso indices. This difference can be explained by different levels of initial ageing, before applying the ageing protocol. It was thought that the plant-mix was more aged initially and then exhibits lower ageing slopes due to a global decrease of the ageing slopes during the ageing protocol. According to the idea that the plant-mix would be initially more aged than the lab-mix, it was decided that a laboratory (here lab 05) could age the plant-mix without the short-term protocol. It seems relevant to consider that the plant-mix has

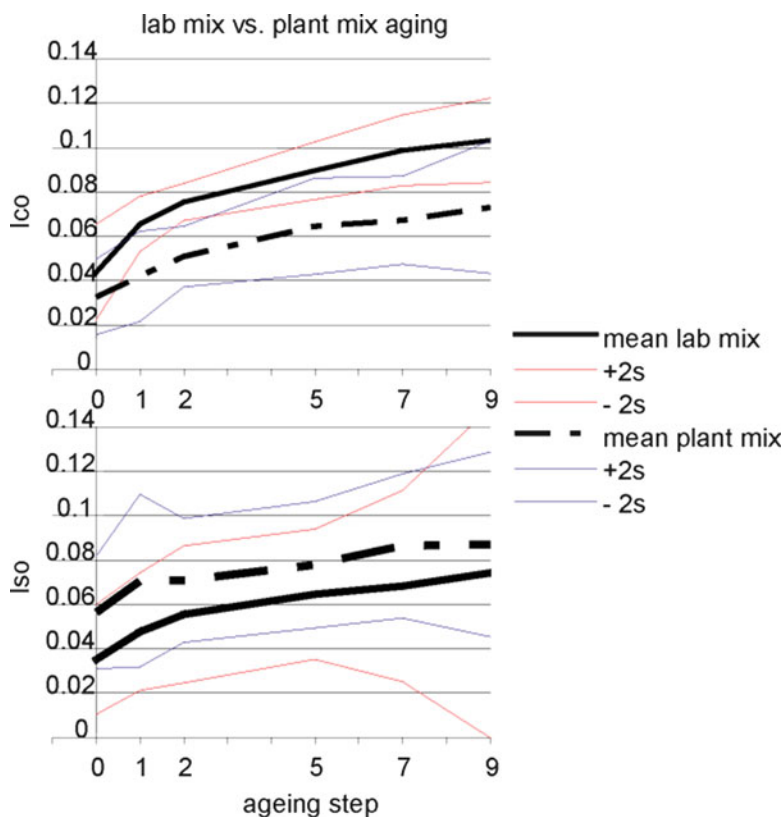


Fig. 7.42 Comparison of the evolution of ageing of the lab-mixes and the plant-mixes

already received short-term ageing during manufacture and transportation. Results are presented in Fig. 7.43. The results of lab 05 are obviously less smooth than the mean plant-mix results. It can be seen that the plant-mix aged without the short-term step still shows less evolution than the plant-mix with the short-term step. For lab 05, the Iso seems not to evolve during the ageing protocol. According to the general scattering of results related to the previous analyses, this lack of evolution is not significant, but it indicates that the evolution may be very low, lower than the evolution for lab-mix and plant-mix with short-term ageing.

A comparison of the scattering between lab-mix and plant-mix results has been performed. For the Ico, the average standard deviation represents 10% of the mean value for the lab-mix and 20% of the mean value for the plant-mix. For the Iso, the average standard deviation (on all the steps) represents 33% of the mean value for the lab-mix and 22% of the mean value for the plant-mix. Thus, it can be concluded that the Ico is statistically more practical to use than that of the Iso one to observe ageing. According to these results, it seems that there is no trend for the lab-mixes to be more scattered than plant-mix. Thus, it is not relevant to consider the manufacturing process responsible for the scattering of results. Extraction and recovery methods seem to be one major clue in the scattering of results.

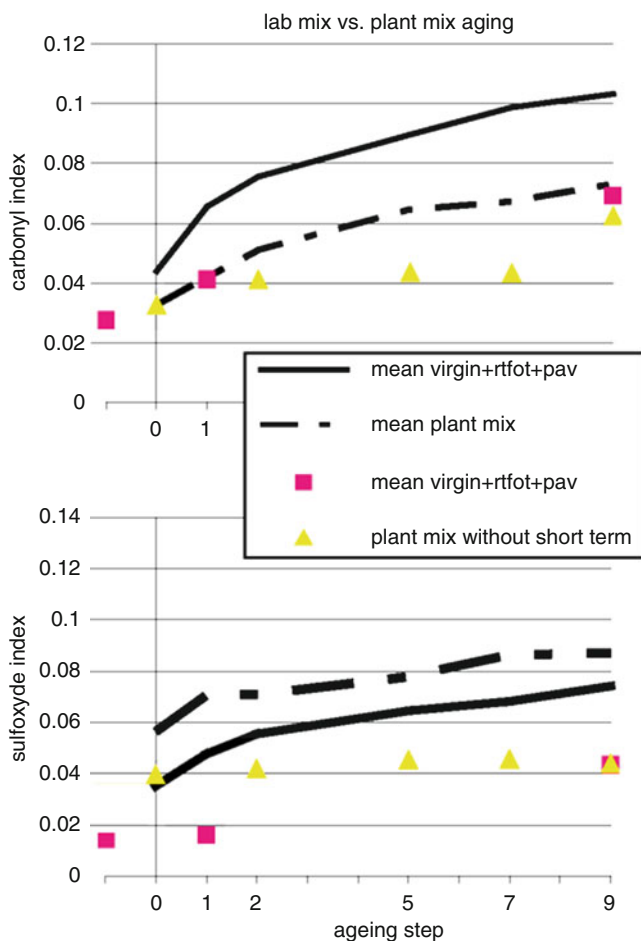


Fig. 7.43 Comparison with a plant-mix aged without the short-term step and with a binder aged with the RTFOT and PAV methods

7.5.3.5 Comparison of Our Ageing Protocol with a Typical Protocol on Bitumen

The results of our ageing test on the laboratory and plant-mixes have been compared to those obtained while ageing virgin bitumen according to the more classical RTFOT and PAV tests. Two laboratories have performed this ageing procedure based on RTFOT and PAV tests. Results are presented in Fig. 7.43.

In this figure, results of RTFOT and PAV tests have been arbitrarily placed at step 1 and step 9, considering that a RTFOT test is equivalent to a short-term ageing

on binders and that a PAV test on a binder is equivalent to a long-term ageing. If we look at the Ico, the RTFOT test is comparable to a short-term ageing on plant-mix but not to a short-term ageing on lab-mix: Ico evolves at 0.83% due to RTFOT, it evolves at 0.93% and 0.21% due to short-term ageing on plant and lab-mix respectively. The Iso does not confirm this trend. The PAV test is comparable to long-term ageing protocols (between steps 2 and 9) on laboratory and plant-mixes, whatever the index.

It can be concluded that the short-term ageing protocol seems to be stronger than an RTFOT test, while the long-term step of our ageing protocol is quite comparable to PAV ageing.

7.5.3.6 Summary of the FTIR Results

Using method 69 for the FTIR measurement to characterise an evolution of oxidation indices is not a very old technique. Thus it is perfected yet. However, some good trends have been seen. First it is possible with such technique, to observe an evolution of oxidation. This evolution is more visible with the Ico than with the Iso one, or due to different kinetics of oxidation, maybe due to different efficiency. The evolution of FTIR results is quite comparable to those for penetration and ring and ball RB-softening point results. The ageing effect tends to decrease along the ageing protocol and the level of ageing of the binder. Results on the plant-mix evolve less than those of lab-mix. The comparison with more classical ageing protocols like RTFOT, PAV and RCAT seems to show that the order of ageing is comparable, but cannot be easily correlated. According to these first results, it was thought that the RTFOT test is more severe than the short-term phase of the protocol, and that the PAV test and the long-term ageing phase have a similar ageing power.

7.6 Conclusions and Recommendations on Laboratory Ageing of Bituminous Mixtures

It has to be pointed out here that all the conclusions are based on the mixture used for this study with a bitumen with a penetration grade of 35.

In this study the focus was on the influence of the binder-aggregate effect in a real mixture on the ageing of the bitumen. The influence of the milling effect was not incorporated, because the ageing was done on a loose mixture.

7.6.1 Conclusions

7.6.1.1 Test Methods

Penetration and RB-Softening Point

After 9 days the penetration value and RB-softening point of the aged samples indicate that the binder still ages from a rheological point of view although the rate of this ageing decreases. So even after 9 days the ageing process still continues.

For penetration the reproducibility of the ageing test does not fulfil the requirements regarding the R-values according to European norms. As mentioned the deviations in the results are the sum of the deviations for mixing, ageing, recovery procedure and the penetration test.

From the statistical analysis on penetration and RB-softening point it can be concluded that it does not make a difference if tests on the recovered binder are done immediately after recovery from the aged material or in a later phase. Time of recovery of the bitumen has only a marginal influence on the test results. Starting the recovery of the aged bitumen after the completed protocol will result in a slightly harder bitumen than doing the recovery immediately after each step in the ageing protocol. This is important for the development of a workable mix ageing method.

The plant-mix, batched in the paver, has already experienced ageing, but no clear difference can be observed for penetration and RB-softening point of the laboratory and plant mixed aged material. After 7 days of ageing with the protocol the values are more or less the same.

Based on the results from the penetration and RB-softening point test on the recovered bitumen it can be concluded that a short-term ageing of 4 h at 135 °C and a long-term ageing of 9 days at 85 °C of the loose mixture, will give reproducible results for ageing of a loose asphalt mixture in an air ventilated oven.

DSR Tests

The DSR tests confirmed the expected changes in stiffness and phase angles both for the laboratory aged mixture and the plant-mix. Systematically the shear stiffness increased with increased ageing time and the phase angle decreased with increased ageing time

From the DSR tests it seems that the plant mixed material gives higher stiffness and lower phase angle than the laboratory mixed material.

FTIR

FTIR results show a good repeatability, but less reproducibility. Ico results are more consistent than Iso results. The comparison of tests performed on the same original

binder has shown considerable differences in FTIR results. Concerning the ageing procedure, it seems that reproducibility may be better when using an offset. A comparison of the FTIR results between laboratory and plant-mixes shows that the plant-mix seems to be less sensitive to ageing. Also a difference between the levels of Ico and Iso for laboratory mixed and plant mixed material is observed. In general, FTIR results confirm the trends given by the penetration and RB-softening point results.

Combined Tests

For all tests the ageing rates are (strongly) decreasing with time of ageing. It seems that in most cases after 7–9 days the rate becomes very slow. For the production of RA in the laboratory time is very important. This conclusion is related to the bitumen used in this research.

7.6.1.2 General Conclusions

From the questionnaire it becomes clear that reuse of RA in hot mix asphalt is a very important development with increasing interest in the addition of high amounts of RA. More knowledge is needed on the effect of high amounts of RA in hot mix asphalt on the properties of the mix. For the research on the ageing protocol it can be concluded that when the shift at the start is neglected, most test results show a similar trend.

The round robin clearly shows the variation that can be expected when testing an ageing protocol at different laboratories. The variation includes mixing, ageing, extraction and recovery of bitumen and test methods. However, taking into account the specified protocol (thickness and volume of material, ventilated oven) and despite the different recovery methods including different solvents, the trends can be considered as consistent.

Comparison with ageing tests on the virgin reference bitumen has resulted in the following conclusions. Short-term binder ageing (RTFOT, RCAT short) is similar to the ageing during production of the mixture in the laboratory (step 0). This means that short-term ageing of the loose mixture after 4 h at 135 °C (step 1) results in higher ageing than the RTFOT ageing on the virgin bitumen. RTFOT + PAV are similar to 3 days of ageing at 85 °C (step 3) in the protocol for the loose mixture. So the laboratory ageing on the binder seems to be quite low compared to the levels reached in the ageing protocol on the loose mixture.

The developed ageing protocol is very suitable to produce a highly aged RA after 9 days. RA at different levels of ageing can be produced in this time period. It is clear from the results that when a certain laboratory consistently uses this ageing method with the same equipment, solvent, recovery method, rheological equipment, FTIR equipment, a very consistent RA can be produced.

In this way, a very practical method is developed to produce RA in large quantities at any level of ageing.

Despite all the work performed, some comments have to be made. With more time, a deeper statistical analysis could be done. It is recommended to perform further statistical analysis on the results with the help of a specialist in this field. Unfortunately, the FTIR results were not always consistent. Part of the problem can be related to the filler still in the recovered binder. More analysis on the results is necessary and extended research is needed to investigate the usefulness of FTIR tests to follow the ageing of the binder.

7.6.2 Recommendations

The conclusions in this report are based on the bitumen used, in this case quite a hard penetration grade bitumen with a penetration of 35. Further studies may be needed to recommend application of the ageing protocol on other penetration grade bitumens, including softer bitumen types and also polymer modified binders.

In this research, plant- and lab-mixes were considered, but no reference to field ageing of this mixture was available to compare the ageing protocol to real field results in time. Only a comparison with PAV ageing could be made and it is clear that the protocol can age the bitumen far beyond the standard procedure with RTFOT + PAV. It is proposed to relate the laboratory ageing procedure to real ageing in field sites.

In this research different solvents are used by the laboratories to recover the bitumen. It is important to research the possible influence of different solvent types, which is going to be done in the Re-road project.

In future research it will be important to know the behaviour of mixtures with RA, before the material is produced in the field. In this case milled or crushed RA material needs to be available for the research. The ageing method as developed provides loose RA material with binder at the right ageing level, but the laboratory RA will need further processing to imitate milled or crushed RA that can be combined with virgin material for the research. It is proposed to develop a procedure to make the loose RA available in milled or crushed form as in practice.

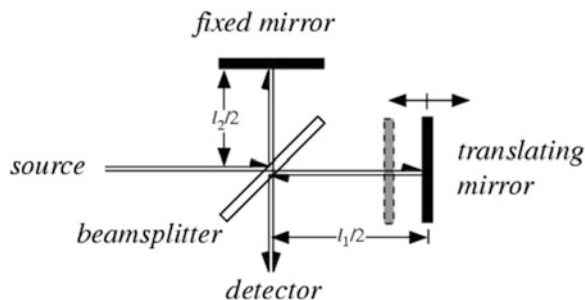
Acknowledgement The authors of sections 7.5 and 7.6 would specially thank Erik Nielsen, Antonio Paez and Patrick Muraya for their valuable comments.

Annex: Principle of Infrared Spectroscopy

Theory of Infrared Spectroscopy Technique

Infrared spectroscopy is an analytical technique that allows identification of functional groups of organic molecules and also certain mineral products. This

Fig. 7.44 The Michelson interferometer [15]



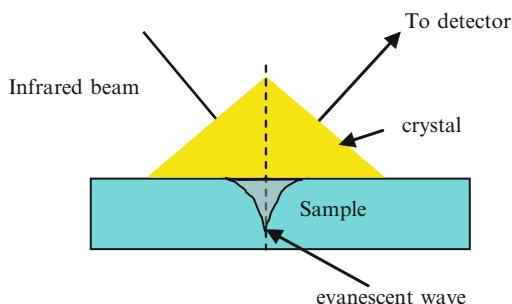
technique is based on the excitation of matter molecules by an electromagnetic wave in the mid-infrared region, approximately $4,000\text{--}400\text{ cm}^{-1}$ ($30\text{--}2.5\text{ }\mu\text{m}$). When a molecule absorbs electromagnetic radiation there is an increase in its total energy. According to a quantum theory, only a quantum of radiation of a specific energy can be absorbed by a molecule, thereby raising its energy from the ground state to an excited state. The energy of transition, ΔE , is equal to the energy of the interacting photon ($h\nu$). The remaining energies of a molecule are due to vibrational, rotational and translational motion associated with the molecule. Infra-red radiation is of sufficient energy to cause transitional, rotational and vibrational energy levels of a molecule. Each vibration mode corresponds to a wavelength or absorption energy. So the analysis of the light-absorption according to the wavelength corresponds to a specific spectrum of a molecule, an ion or a crystallised structure and can be considered as a digital print.

IR spectra are collected using Fourier Transform Infrared (FTIR) spectroscopy. Where radiation is varied by a monochromator, the IR radiation is guided through a Michelson interferometer (see Fig. 7.44). The purpose of the interferometer is to have a beam of IR radiation, split it into two beams, and make one of the beams travel a different (optical) distance than the other in order to create alternating interference fringes (Smith [29])

The FTIR spectra are usually presented as plots of intensity versus wavenumber (in cm^{-1}). Wavenumber is the reciprocal of the wavelength. The intensity can be plotted as the percentage of light transmittance or absorbance at each wavenumber.

The infrared spectra can be obtained in different modes (transmission, Attenuated Total Reflectance (ATR) etc.). In this work, the transmission mode was used by seven laboratories and the ATR mode was used by one laboratory. In the transmission mode, IR radiation is passed through the sample. Some of the infrared radiation is absorbed by the sample and some of it passes through (transmitted). Attenuated total reflection infrared (ATR-IR) spectroscopy is a subset of Infrared spectroscopy which enables samples to be examined directly in the solid or liquid state without any preparation. ATR spectroscopy and multiple internal reflection spectroscopy are increasingly gaining popularity as significant tools to analyse coatings and opaque liquids. In the ATR mode, a beam of infrared light is passed through the ATR crystal

Fig. 7.45 Schematic a simple reflection ATR system



which is covered on the top by the solid or the liquid sample such that the incident infrared light reflects internally after coming in contact with the crystal covered with the sample, forming an evanescent wave which extends into the sample, typically by a few microns. The angle of incidence must be greater than the critical angle for total internal reflectance of the infrared light. When the beam exits the crystal, it is collected by a detector and analysed and displayed in form of the ATR-IR spectra (Fig. 7.45). The ATR spectrum is simply the IR spectrum, so all the typical uses apply.

Quantification of Species

FTIR analysis allows the quantification of a given function by applying the Beer Lambert law:

$$A(\nu) = \varepsilon_i(\nu) \times l \times C_i \quad (7.2)$$

with:

$A(\nu)$: absorbance for the wavenumber ν

$\varepsilon_i(\nu)$: molar absorptivity coefficient for the wavenumber ν

l : optical path length

C_i : studied species concentration

In the case of complex systems, like bitumens, the superimposition of absorption bands, due to the absorption of several molecules, is frequent. It is so possible overall to integrate several vibrations of the same type² (for example, the C=O ester, acid, ketone, lactone vibrations between 1,800 and 1,635 cm^{-1}), by considering an area comprised between two wavelengths.

²The means of signal processing (derivative, deconvolution, simulation) allow to detect the components of such bands.

On the basis of Beer Lamber law, the global absorbance can be then written as follows;

$$\int_{\nu_1}^{\nu_2} A(\nu) d\nu = \int_{\nu_1}^{\nu_2} \varepsilon_i(\nu) \times l \times C_i d\nu \quad (7.3)$$

with :

$A_{(\nu)}$: absorbance for the wavenumber ν

$\varepsilon_{i(\nu)}$: molar absorptivity coefficient for the wavenumber ν

l : optical path length

C_i : studied species concentration

But, for following the absorption area evolution or for the comparison of two samples, it is important to always consider the same band widths.

Application of FTIR to the Oxidation State of Bitumens

Apart from the fact that FTIR is useful for the detection and analysis of chemical oxidation bands (Doumenq [7, 8]; Pieri [22]), it can be also used for the classification of bitumen's composition in terms of functionalities (aliphaticity, aromaticity, substitution etc.), that are quantified by indexes, expressed as ratios of characteristic spectral absorption bands (Pieri et al. [23]).

The assessment of oxidation level has been performed according to the test method of LPC no 69.

Preparation of Test Samples

Bitumen samples are prepared according to the norm NF EN 12594 ([2]). The binder is heated at a temperature high enough to decrease its viscosity before being stirred with a spatula. The quantification of oxygenated species is performed using the analytical technique of FTIR spectroscopy in transmission mode according to two possibilities of sample preparation:

- A hot binder droplet is laid on a thin plate transparent to the infrared radiations (KBr, NaCl or CsI) and then it is spread with a spatula in order to obtain a sample film transparent to the infrared radiation.
- A binder solution in the dichloromethane of "spectrosol" quality is prepared at 30 g/L concentration from 100 mg of sample intake (Pieri [22]) or at 170 g/L concentration from 500 mg of sample intake (Durrieu et al. [9]). These solutions are then directly analysed by FTIR spectroscopy according to the so-called dry-plate technique (Smith [29]), consisting in laying such a quantity of solution (25 μ L) on an infrared thin plate (KBr, NaCl or CsI) and then to evaporate the solvent in order to obtain a homogeneous thin film of sample transparent

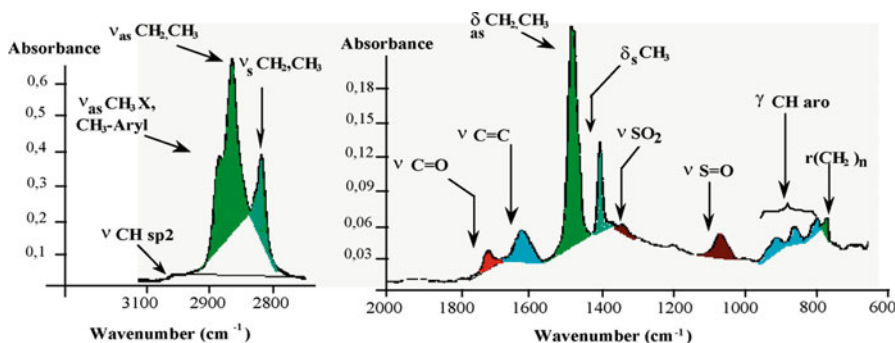


Fig. 7.46 An example of FTIR spectrum of bitumen recorded between 4,000 and 400 cm^{-1}

to the infrared radiation. The total evaporation of dichloromethane is checked by the absence of infrared band at 1,265 cm^{-1} in the obtained FTIR spectra. In transmission mode, the spectra are recorded between 4,000 and 400 cm^{-1} with a scans number of 32 and a resolution of 2 cm^{-1} .

For these two techniques of sample preparation, it is important to verify that the obtained signal is in the concentration's domain that permits to obtain a linear response of measured areas and, consequently, to lead to a constant value of indexes whatever bituminous binders are analysed.

An example of an obtained spectrum is shown on Fig. 7.46. The effects of bituminous binder's ageing by oxidation are well-known in relation to the apparition of oxygenated chemical species. As a result, only the carbonyls functions (CO showing a characteristic absorption band around 1,700 cm^{-1}) and sulfoxides functions (SO showing a characteristic absorption band around 1,030 cm^{-1}) are selected.

Interpretation of FTIR Spectra of Bitumens and Calculation of Oxidation Indices

In order to take account of the majority of oxygenated species of bitumens, the calculations have been performed on an area of absorption bands. It is important to note that, in case of the follow-up of ageing evolution by oxidation, the formed oxygenated compounds that absorb out of selected boundary markers will be not taken into account.

Despite the sample preparation being done carefully, it is impossible to obtain a binder film with a perfectly homogeneous and reproducible thickness. So, it becomes difficult to obtain data in relation to molar absorptivity or extinction coefficients ϵ of each absorption band characteristic of various functional groups (these ϵ depend on the studied chemical group, the electronic environment of the group and the nature of the sample itself) and to determine the optical path length (thickness of the sample) as well.

In order to remove problems linked to the determination of the optical path length and molar absorptivity or extinction coefficients for complex chemical systems, the calculated areas are used in determining band area ratios characteristic of the relative proportion of the different chemical species (namely aromatic CH/aliphatic CH, etc.) or of the relative proportion within the sample (namely aliphatic CH/sum of integrated band areas, etc.). This methodology, called semi-quantitative infrared analysis (Doumenq [7, 8]; Ganz and Kalkreuth [11]; Pieri [22]; Mouillet [17]), allows to disregard the thickness of the sample.

By considering two functional groups A and B, the ratio of the corresponding infrared band areas is defined, based on the Beer-Lambert law, as a function of ratio of the concentrations of the two studied species, increased in ratio of the two extinction coefficients.

$$\text{Area (A)}/\text{Area (B)} = \varepsilon(\text{A})/\varepsilon(\text{B}) \cdot \text{C(A)}/\text{C(B)} \quad (7.4)$$

Assuming that the changes of the ratio $\varepsilon(\text{A})/\varepsilon(\text{B})$ is small within a same sample's nature (for example, from one bituminous binder to another), the area ratios can therefore be compared relatively from one spectrum to another, in order to characterise the change of concentrations ratio (Ruau et al. [28]). Assuming that the ethylene groups (CH_2) at $1,460 \text{ cm}^{-1}$ and methyl (CH_3) at $1,375 \text{ cm}^{-1}$ are not significantly altered by oxidation of binders, the oxygenated functions quantities are determined using a semiquantitative approach based on the definition of the two following structural indexes:

- Carbonyl Index :

$$\text{Ico} = \frac{\text{Area around } 1,700 \text{ m}^{-1}}{\text{Area around } 1,460 \text{ cm}^{-1} + \text{Area around } 1,375 \text{ cm}^{-1}} \quad (7.5)$$

- Sulfoxide Index:

$$\text{Iso} = \frac{\text{Area around } 1,030 \text{ cm}^{-1}}{\text{Area around } 1,460 \text{ cm}^{-1} + \text{Area around } 1,375 \text{ cm}^{-1}} \quad (7.6)$$

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Chapter 8

General Summary and Outlook

Emmanuel Chailleux and Manfred N. Partl

Abstract Final considerations based on the experience and feedback of this RILEM technical committee TC 206-ATB on “Advanced Testing and Characterization of Bituminous Materials” are given. Some open questions within the general methodology as introduced in Chap. 1 and used as guideline for RILEM activities are listed. Future prospects and direction of activities of joint voluntary research efforts within the frame of such asphalt-related committees are also discussed. It is suggested that these future efforts will most certainly have to focus even more on sustainability and environmental aspects but without neglecting general materials and systems quality aspects during production and construction as well as safety and performance during use. Future activities will also have to deal with multi-functionality aspects of pavements and multi-scaling approaches as a basis for better understanding and tailoring of pavements and pavement materials.

Keywords Pre-standardization • Future activities • Bituminous materials • Sustainability • Performance • Multi-scale • Multi-functionability • Harmonization

8.1 Introduction

Road construction is different from other production industries because it has to be carried out with moving factories under often rapidly changing outdoor environment as well as locally varying soil and topographic conditions. This means that

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it is impossible to produce several prototypes before commercializing a product and that each pavement is a unique individual specimen. Hence, uncertainty in the quality of the end product may be quite high adding to the risk of not meeting the high expectations of the customers who require a safe, smooth, durable and maintenance-free driving platform for ever increasing loads and traffic frequencies.

Unfortunately, in practice, due to increasingly efficient paving technology, it may well happen that insufficient product quality is detected to late, i.e. only after the flaw has already been reproduced for many kilometers. Adding to the challenge is the fact that most of the time, materials choice is limited, especially concerning aggregates which have to be selected close to the work site. Hence, reliable pavement design methods become extremely important. In order to optimize the design, materials have to be tested under performance related conditions for providing a realistic link to the actual field situation.

From the previous chapters follows that the activities and interlaboratory tests conducted by RILEM TC 206-ATB on advanced testing and characterization of binders and mixtures produced new results and insight regarding accuracy and relevance of different test methods for assessing performance related properties of bituminous road materials. It is expected that these results will contribute to improve comparability of test results on an international level and also serve as input for standards in the sense of pre-standardization.

8.2 RILEM Technical Committee Summary and Feedback

In the five technical groups (TG) of the RILEM technical committee 206-ATB, interlaboratory tests were organized based on the general methodology for testing of bituminous pavement materials and following the principles for planning interlaboratory tests presented in Chap. 1. Each case was aiming at determining a specific performance property or material characteristic as useful practical information for bituminous pavement construction presented in Chap. 1. This type of cooperative work on a fully voluntary basis was a true challenge since it involved laboratories and partners from numerous countries, with different scientific or engineering cultures, objectives and point of views. Due to the common language which was a necessary basis for trying to answer the same problem, work performed on the same objects, was very informative regarding test methods on bituminous materials.

TG1 was dedicated to the measurement of bituminous binder fatigue performance in comparison to mixes (see Chap. 2). A round robin test was carried out to assess a procedure using Dynamic Shear Rheometers (DSR). This procedure appears now to be applicable by most of the existing DSR and a strong link has been found between the binder fatigue performances and the corresponding mixes. Nevertheless, in a preliminary part, reproducibility problems were encountered. Temperature control in the DSR equipment was generally the lonely source of wrong measurement. Since it is generally well known that temperature is a first order parameter for bituminous materials; this parameter remains the main point

to control. Regular interlaboratory tests and equipment with auto-control are necessary. The results also underline that equipment has to be chosen carefully knowing that DSR manufacturers have not all the same experience in testing bitumen.

TG2 focused on asphalt mixture compaction method as well as measurement and quantification of aggregate structure characteristic (see Chap. 3). It conducted an extensive literature review on imaging techniques showing that these techniques are promising add-ons to conventional volumetric and mechanical characterization methods. In particular, it was concluded that a simple standard imaging and analysis tool with simple digital scanner was a possible option to help mixture designers to better optimize asphalt mixture design, and possibly control its quality. Hence, simple software was developed and utilized in a comparative study with several laboratories, demonstrating its user friendliness and effectiveness for quantifying variation of aggregate structure for a specific compaction method. A draft standard document and new parameters for defining aggregate structure in terms of orientation, special distribution, and contact points were proposed. It is expected that this effort will encourage others to utilize this tool to advance understanding of asphalt mixtures and improve mixture design procedures in the future.

Rutting phenomena were the subject of the TG3 (Chap. 4). Several aspects were treated, from both an experimental and modelling point of view. Although the three wheel tracking testers (WTT) gave the same ranking on three different mixes, rutting performance appeared to be not strongly linked to field behaviour. Indeed, even the most commonly used WTT in this interlaboratory test (the French wheel tracking tester) was not able to correlate with the good field performance of a specific bituminous mix observed after 10 years of service. The method seems to be generally too severe, which could be very negative for the development of some new products. A new test or a new standard may be needed. In addition to this interlaboratory test, a subgroup has worked on modelling of the wheel tracking test, taking into account the viscoelastic behaviour of the material. This theoretical approach was carried out in several laboratories, using different constitutive laws and finite element codes. It was demonstrated that analogous to interlaboratory experimental tests, modelling procedures must be defined precisely. This is true for rheological properties calibration as well as finite element code implementation.

TG4 work has been focused on both the final evaluation of a long term pavement performance prediction evaluation on a test section in Austria (Chap. 5) and the assessment of interlayer shear properties measured through interlayer shear resistance tests (see Chap. 6).

In case of the long term performance prediction for the Austrian Test section it was concluded that traffic and temperature data can be collected and predicted quite precisely whereas damage predictions based on laboratory experiments and theoretical analysis are difficult due to the great variety of test methods and prediction models.

As for the interlayer shear properties, the working program included three type of interfaces from real test sections, five different tests and several testing conditions such as stress, temperature and sample geometry. This huge program allowed determining the influence of each parameter including repeatability and

reproducibility and finally establish relations between test methods. It also allowed finding conditions for which the shear resistance of the interface is evaluated and not just the resistance of the layers. Data from this study will be very useful for standardization. This work underlines that it is necessary to have a complete and carefully planned testing program with well-defined material in order to reach practical output.

TG5 works dealt with recyclability of mixes, aiming in a first step at controlling rap production in the laboratory (Chap. 7). The idea was to get a laboratory methodology for assessing ageing of bituminous materials in order to forecast the “recyclability” of the mix. Two simple and easily reproducible ageing methods have been defined on loose mixture: one for short term ageing and another one for long term ageing. A round robin test with these ageing protocols was organized. It included infrared spectroscopy, penetrability, ring and ball tests and complex modulus measurements on the recovered binder. Although all the laboratories found the same relative variation of physical and chemical properties, a gap was systematically observed between absolute values. At the moment, it is difficult to determine precisely the reason for this scattering. It seems that extraction or/and sampling methods should be harmonized.

8.3 RILEM Technical Committee Outlook

8.3.1 Main Research Directions and Challenges

Knowledge on bituminous material increases day after day. Hence, for the next step in RILEM technical committee activities, it will be necessary to adapt the general methodology for testing of bituminous pavement materials presented in Chap. 1 to a new context, in strong relationship with sustainable development and condition of use. This means also dealing with new or non-conventional materials as well as new procedure in the laboratory linked to new construction and asphalt production technologies. Indeed, today’s context requires considering the following key aspects:

- Saving of material resources and energy
- Increase of the ratio durability/cost, life cycle assessment LCA
- Reduction of negative environmental effects – CO₂ pollution
- Increase of long-term safety
- Enhancement of multi-functionality of pavements

Each of these items can be translated into research needs concerning tests or procedures on multi-scales, i.e. from nano- to macro-scale [6, 10], even involving accelerated pavement testing APT with traffic load simulators of different types and sizes [7, 8].

However, it is crucial to understand, that such research should not be performed in exclusive stand-alone initiatives but, whenever possible, in joint efforts in order to reduce the risk of errors, duplication and waste of resources in a very early stage. It should also be done by building on existing knowledge, methods and models following general scientific principles and keeping in mind the prospective of practical application and standards. In particular, the development of a new test method means to make sure that this test method is sufficiently validated with regards to practice and the general application limits of the answer that is expected from this test. It also implies that a sound basis has been established with respect to repeatability and reproducibility data, e.g. through interlaboratory tests following clear guidelines such as presented in Chap. 1.

Providing thorough information on the range of validity and application as well as careful evaluation of errors and influence of experimental setup is imperative, but, unfortunately not always respected, due to time, financial, priority and human restrictions. In addition, experience shows that there is often a tendency and temptation to apply test methods in an extrapolating way for cases, where these tests were originally not designed for without accounting for the consequences and without awareness of the real physical, mechanical or chemical background of the test method. The result of this situation is an unnecessary variety of uncoordinated and overrated test methods creating confusion and uncertainties worldwide.

8.3.2 Saving of Material Resources and Energy

Most of industries face the lack of natural resources which become more and more difficult and expensive to exploit and import. Nowadays, being thrifter appears to be beneficial for environmental as well as for economic reasons. Pavement industries have to answer to this problematic in conjunction with political questions. In terms of material resources, availability of aggregates and bitumen is of main concern, but also energy consumption and CO₂ balance during the manufacturing process are also important issues.

In densely populated areas, opening new quarries becomes more and more difficult. Concerning bitumen, some questions, linked to oil production, arise. Regarding this context, use of more reclaimed materials appears to be necessary. It means that new test methods will be needed, as initiated by TG5 (Chap. 6). In the future, with regard to reclaimed materials, the following methodologies deserve further development:

- Methodology aiming at defining how to prepare a recycled mix in lab comparable to what happens in plant (e.g. overcoming the challenge of scaling and the influence of different mixing energy).
- Method to understand and assess a rejuvenating agent according to the physico-chemical mode of action of the additive.

- Method to assess compatibility and interaction between old and new bitumen. To do this, it will be necessary to better understand and model the physical and chemical mechanisms of blending.

Recent technological developments have been carried out to reduce fuel consumption during the mixing process in the plant. Warm, half warm and cold mixes differ considerably from classical hot mix asphalt in terms of materials employed and processing. In order to be relevant for the actual field situation test methods which take into account the following particularities are needed:

- Methods for measuring workability of the mix, given the fact that different physical mechanisms may be involved, such as viscosity of the binder or/and lubrication of the contact zone between aggregates and bitumen.
- Methods to reproduce half warm and warm mixes in the laboratory, taking into account the different processes used in plant.
- Concerning cold mixes prepared with emulsified or foam bitumen, strong efforts are also needed to reproduce fabrication in the laboratory in terms of mixing process, curing condition and measurement of mechanical properties. Again, a fundamental approach will be needed to understand the physico-chemical phenomena involved in this case.

New products from biomass have been developed since 10 years [1] and others are under development [5, 12]. These “green” binders cannot be characterised as classical bitumen. For this type of materials, the task would be to re-define the set of characterization methods and criteria which may be used for all type of binders. This characterization has to be performed by intrinsic measurement, requiring:

- Methods to assess ageing kinetics in the laboratory in relation to “natural” ageing in the field
- Methods to measure rheological properties in the linear and non-linear domain as compared to classical oil-based bitumen

8.3.3 *Increase the Ratio Durability/Cost, LCA*

Private or public road management authorities are in constant need of optimizing and reducing costs of construction, long term maintenance and renaturation, considering the whole lifetime “from cradle to grave”. From an entire society point of view, these costs should not only be related to the pavement itself but also to additional costs that are created from lost working hours due to congestions and other factors. Increase of infrastructure lifetime may be beneficial as soon as investment is counted on long period. Choice of materials at the beginning of the construction is particularly important. However, specifications or methods used today seem not always adapted to help designer to choose the best materials regarding durability. Further research efforts have to be undertaken to elaborate:

- Methods to determine and assess real performance of modified bituminous materials. Some tools used today are not suited to highlight good performance of certain materials although good field behaviour has been observed. This holds for example with respect to polymer modified bitumen and cracking properties, hydrated lime modified mixes and ageing resistance, etc.
- Methods to characterize mixtures during artificial ageing in relation to in field aging. There is a need to develop ageing protocols with UV and moisture associated conditioning that make also use of non-destructive tests.
- Methods that provide answers in special cases where the loading condition and the damage modes are different from a classical road, e.g. in case of roundabouts, bridges, bus lanes.

8.3.4 Environmental Effects – CO₂ Pollution

In addition to mechanical properties, environmental performance needs also to be measured, quantified and predicted. Some particular questions arise concerning the manufacturing process, especially with new materials. Nowadays, it is difficult to assess in a scientific, generally accepted consensus way if a new process or product is really environmental friendly. Hence, there is a need for:

- Methods to capture, measure, and quantify the chemical substances which can be found in fumes during the mixing process. For this, special protocols in the laboratory have to be defined in order to test the effect of different parameters, such as temperature, additives, type of bitumen.
- Methods to measure the energy needed by a process in order to evaluate the global CO₂ equivalent gain.
- Methods to evaluate emissions during paving, especially with respect to extruded water from emulsion and additive leaching.
- Methods to measure noise generated by different surface courses

8.3.5 Safety

Safety of road user is generally one of the priorities of road governmental agency. Regarding bituminous road materials, safety is linked to friction coefficient and visibility. However, nowadays accepted tests which may cover these aspects in an internationally recognized and universal way, are still missing. With regard to skid resistance activities should concentrate on the interlaboratory evaluation of:

- Methods to assess friction coefficient and evolution of skid resistance as a function of surface temperature, surface roughness, climate and aging.
- Laboratory and field tests to determine and model skid resistance and tire pavement interaction in a performance related way over the whole life time

- Methods to assess visibility, e.g. during rain (splash and spray) and during the night, including long-term effects.

8.3.6 Multi-functionality

Asphalt roads are covering significant areas that may also be used for different purposes in addition to their traffic carrying function, such as noise absorption, water retention and drainage, air and water cleaning and energy harvesting, heat transport from urban heat islands, etc. However, the roads of today are generally not designed for or of sufficient quality to allow such multi-functional applications. Future research activities will also have to deal with multi-functionality aspects of pavements and multi-scaling approaches as a basis for better understanding and tailoring of pavements and pavement materials. Developing durable, multi-functional road requires new special methods:

- Methods to measure the effect of additional functions as basis for materials development, considering physical, chemical and mechanical aspects in a multi-scale range
- Methods to assess the additional functions and their combinations in the laboratory and in the field in relation to their traffic performance also considering long term effects

8.4 Concluding Remarks

In most industrial countries, the design of materials and structures for road construction originally followed internal rules defined by each road authority individually. However, since more than 20 years, there is a tendency to unify these rules at a larger scale, like in the United State with the SHRP program and in Europe in conjunction to the European standardization.

This development did not happen without any problems, though. For example, the harmonization of the European standards has clearly demonstrated the difficulty to define a common approach. Indeed, this experience has shown that materials and methods are different from one country to another and that the methods are therefore often linked to a specific type of material, tradition or a special context.

Hence, the transposition and harmonization of test methods may turn out to be a tiresome and painstaking process that has the potential to create confusion. The TG3 work, mentioned above (see also Chap. 4), is an illustration of this problem. Mastic asphalt is widely used in northern Europe and shows good performance. This type of bituminous mix which follows the skeleton concept (shown in the Fig. 1.1 of the Chap. 1) is not used in other European country such as France. In fact, French bituminous mixes are traditionally designed following the packing

concept [2]. Therefore, during the TG3 interlaboratory program (Chap. 3), the French wheel tracking tester has classified a specific mastic asphalt as bad material whereas field behavior after 10 years appeared good. This example shows that methods developed and validated in the past for one type of materials, cannot be a priori generalized. It is clear that more general and relevant test methods are needed. RILEM methodology, as it is defined in Chap. 1, intends to meet this challenge, using rational approaches shared by a large international community of engineers and researchers.

The RILEM technical committee 206-ATB, has worked in this direction [9], following simple but decisive guidelines for performing interlaboratory tests (Chap. 1). Results from the different TGs provided information and recommendations about test method based on a validation process ranging fundamental tests in the laboratory to validation and practical performance evaluation in the field. The interlaboratory study of the different compaction methods in the laboratory used in different countries (Chap. 3) produced valuable results not only from a practical point of view but has also shown that fundamental characterization using computer tomography and image analysis is necessary for enhanced understanding of the effect of the compaction on microstructure.

RILEM research and pre-standardization activities, although completely performed on a voluntary basis and driven almost exclusively by the scientific interest and curiosity of the individual members, may indeed provide valuable contributions to the engineering community dealing with bituminous materials [3, 4, 11, 13]. The main advantage of participating in these activities is the possibility to define and compare methods on an international scale, taking into account different scientific and engineering cultures, objectives and point of views. The systematic interlaboratory tests performed in each TG can certainly be considered as positive steps towards a precise assessment of methods and materials.

In spite of the fact, that future RILEM activities will have to take place in the context of sustainable development, classical durability issues will still play an important role and will become even more important under difficult economic conditions.

In order to make progress in the characterization and validation of bituminous materials, a more general approach should be undertaken also taking into consideration multi-functionality issues and multi-scale investigations. This means evaluation of test methods for measuring and characterizing performance and physico-chemical effects with respect to new functions. It also means evaluating test methods and models in the nano- and macro-scale taking also advantage of sophisticated equipment, such as synchrotron, computer tomography and traffic simulators for accelerated pavement testing. Macro-scale investigations should also involve the evaluation of in-field measurement techniques for layer density, moisture, stiffness, aging, cracking resistance and wave propagation properties as well as pavement deflection and acceleration, tire-pavement interaction, noise reduction, optical properties etc.. For this reason, collaboration on an international level is essential and should be intensified with other technical committees, in particular those already existing on bituminous materials dealing with the topics of cracking mechanisms and nanotechnology as well as others yet to come.

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