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Use of Reinforcement Grids for Pavement Maintenance

Utilisation de grilles de renforcement pour l'entretien des chaussées

Verwendung von Bewehrungsgittern für die Erhaltung des Strassenoberbaus

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Research project VSS 2019/422 at the request of Schweizerischen Verbandes der Strassen- und Verkehrsfachleute (VSS)

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List of abbreviations

4PB	4-Point Bending
ARAN	Automated Road Analyser
Art.	Article
AST	Advance Shear Test
ASTRA	Bundesamt für Strassen
BFH	Bern University of Applied Sciences
CAE	Computer-aided Engineering
DIC	Digital Image Corelation
E.g.	For Example
Empa	Swiss Federal Laboratories for Materials Science and Technology
Ex.	Example
FEDRO	Federal Roads Office
FEM	Finite Element Method
FWD	Falling Weight Deflectometer
GUI	Graphic User Interface
IMC	Infrastructure Management Consultants
KPI	Key performance indicator
LCA	Life Cycle Assessment
LCCA	Life Cycle and Cost Analysis
LVDT	Linear Variable Differential Transformer
MMLS3	Mobile Model Load Simulator
MPD	Mean Profile Depth
Re.	Regarding
Vol.	Volume
NBFT	Notched Beam Fatigue Test
FGSV	Forschungsgesellschaft für Straßen- und Verkehrswesen
NMAS	Nominal Maximum Aggregate Size
SAMI	Stress Absorbing Membrane Interlayer
AC	Asphalt Concrete
EN	European Normen
SN	Schweizerische Normen
OpenCV	Open Computer Vision
API	Application Programming Interfaces
OpenMP	Open Multi-Processing

Summary

Introduction

Asphalt pavements exposed to heavy traffic and weather conditions reach their bearing capacity limits over time and show various types of damage (cracks, ruts, etc.) that affect the safety and comfort of road users. Therefore, it is important to seek efficient maintenance and repair methods that make it possible to reduce the cost of maintenance and increase service life, as well as to save materials and energy. Diverse types of measures are available for the maintenance of road infrastructure, whereby reinforcement techniques are becoming popular especially to prevent premature cracking. Although international literature shows that in general, asphalt reinforcements have a positive effect on pavement performance, there are still many open questions, especially regarding their effect on interlayer bonding. In addition, the ASTRA 2011/011 study provided very interesting preliminary results on the use of grids and the associated performance improvements (strength and durability). Grid manufacturers, for their part, have conducted extensive research on their materials and have developed considerable expertise, enabling them to refine their solutions and successfully implement their products on construction sites. However, these studies have been conducted independently, utilizing different methods and tests. As a result, the findings are typically only applicable to the specific product being examined.

Although the effect of reinforcement grids on asphalt layers has been proven in practice in many cases, there is currently no reliable theory to describe the mechanisms and effectiveness for their use. Hence, there is a need for meaningful investigation and testing methods that enable a direct comparison of different reinforced asphalt layers as well as their overall effectiveness and impact on pavement performance.

In order to close this gap, the current research project evaluated the performance of different reinforcement grids using various laboratory tests based on different specimens' sizes and dimensions gaining insights into the use of reinforcement as best practice. In parallel, a field test track was constructed to monitor and link the field performance of the reinforcements to the laboratory evaluation.

Objectives

The project aimed to investigate and promote best practices for the use of reinforcement grids in bituminous pavements by providing the methodological and normative basis necessary to consider them in a design process and to evaluate their effectiveness (performance). The general basis of the project was not related to a specific product. Therefore, the main suppliers of reinforcing grids operating in Switzerland were considered, with the results presented anonymously.

Methodology

The methods applied in this project were based on laboratory and field investigations along with a life cycle cost analysis of the results. The main laboratory investigations were carried out at Berner Fachhochschule (BFH) and Empa. To produce the required specimens for the laboratory investigations at BFH, the production information was documented and then a practical demonstration of the grid installation method of the respective grid providers was carried out at BFH's laboratory. Finally, the test specimens were produced according to the specified instructions. For the larger-scale specimens tested at Empa, the grid suppliers were invited to install their products. All laboratory specimen preparation was conducted in accordance with the requirements and guidelines provided by the asphalt suppliers, using the materials they supplied.

The different stages of the project investigation are presented in Figure 1. After an update of knowledge and the study of recent national and international literature (Task 1), an experimental investigation was conducted in the laboratory (Task 2). These investigations included, among other things the evaluation of grid/asphalt composites (adhesion between layers) and the investigation in terms of fatigue strength and cracking (4-point bending and traffic simulator experiments) as well as overlap length evaluation. The full-scale test Sections (Task 3) focused on the practical aspects of grid installation and monitoring of their in-situ performance regarding deterioration due to mechanical and climatic stresses under real traffic. For this purpose, field test with distress measuring campaigns surveys included Automatic Road Analyzer (ARAN), Falling Weight Deflectometer (FWD) measurements were carried out regularly and core were taken and analyzed in the laboratory. The various data collected in the laboratory and in situ were used to refine and update interlayer reinforced pavement modelling (Task 4). Finally, a life-cycle cost analysis (LCCA) incorporated the findings from all stages of the project (Task 5). This analysis contributed to the development of a comprehensive methodology and detailed recommendations for the design and implementation of maintenance and rehabilitation activities involving reinforcement grids. By integrating lessons learned throughout the project, Task 5, ensured that conclusions were practical and applicable for the design and implementation of maintenance/rehabilitation activities incorporating reinforcement grids while considering the in-situ conditions (climatic conditions, condition of the receiving layer, installation conditions, etc.).

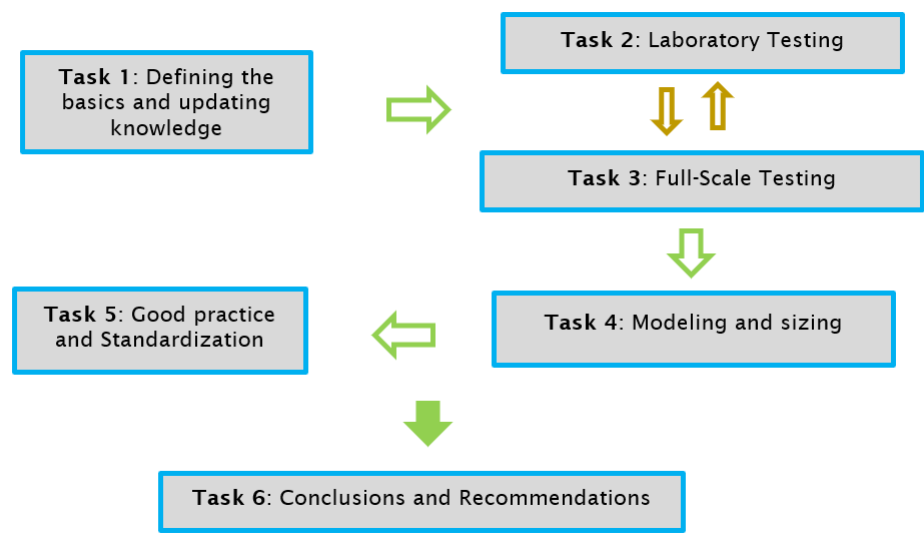


Figure 1: Different stages of the project (Figure 6 in report)

Materials

For all tests carried out in this project, the asphalt concrete mixtures of AC 8 S (surface course) and AC B 11 S (base course), were used. The mixtures were obtained from an asphalt plant in Hinwil. The tack coat used for interlayer bonding was a polymer modified cationic emulsion with a residual bitumen content of 60 %. System C includes a self-adhesive membrane and therefore doesn't need tack coat.

Four different kinds of reinforcement grids were used. The summary of the most relevant mechanical properties is shown in Table 1. Ensuring the anonymity of the results, the grid products are labelled with the letters A, B, C and D.

Reinforcement grids								
Product	A		B		C		D	
Tack coat-Field (residual) [g/m ²]	180		690		none*		420	
Tack coat-Lab (residual) [g/m ²]	180		480		none		250	
Strand direction	L	T	L	T	L	T	L	T
Strand Material	Glass	Carbon	PET		Glass		Glass	Carbon
Mesh size [mm]	15	15	40		10		18	18
Tension strength [kN/m]	120	200	50		100		120	200
Elongation [%]	3	1.5	12		3		3	1.5

*Emulsions was only applied on the edges and top of the grid (600 g/m²)

Table 1: mechanical properties of the reinforcement grid used in the project (Table 2 in report)

Results

Interlayer bonding -laboratory

The laboratory investigation involved the evaluation of mixture specimens prepared at different scales. Double layer slab specimens were prepared at Empa and BFH for the evaluation of the layer adhesion and crack propagation. Cores were taken from the constructed slabs for the investigation of layer adhesion using standard Leutner and Advance Shear Test (AST).

The evaluation of the interlayer bonding using the Leutner test performed on laboratory specimens showed that almost all the reinforcement systems, except for the A, have a negative influence on the interlayer bonding. In the laboratory investigation, the maximum shear forces of the investigated reinforcement systems differed quite significantly. Especially, System A achieved with an average of 33.2 kN very high values, followed by System B with an average value of nearly 25.8 kN, while System C performed very poorly with an average max. shear force of less than 6.0 kN. System D reached an average of 16.0 kN but considering the standard deviation the required value of 15 kN according to the Swiss standard (SN-EN-12697-48) could not be achieved for all tested specimens. System A even received higher values than the unreinforced system (reference) which achieved an average max. shear force of 30.3 kN. The results are visualized in Figure 2.

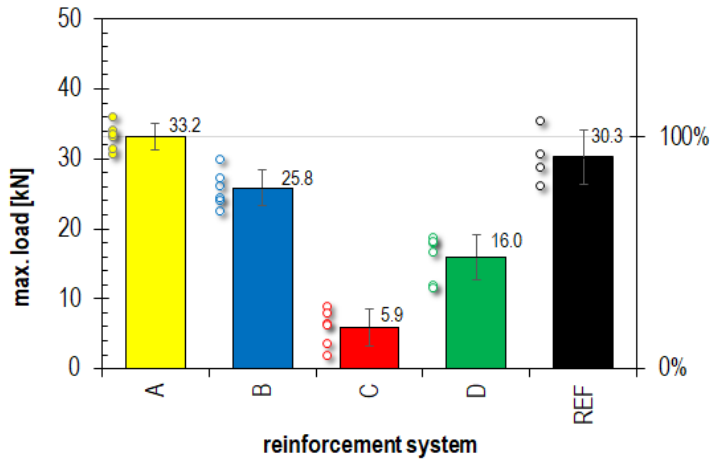


Figure 2: Leutner test laboratory results for each reinforcement system (Figure 15 in report)

The laboratory test results using AST test showed that the layer adhesion of the reference specimen has a higher resistance against the interface fracture in comparison to the test specimens with reinforcement at the interface. As shown in Figure 3, the specimens with grid product A have the highest resistance against interface fracture in comparison to the other reinforced specimens. Furthermore, after water conditioning product A has higher resistance against the interface fracture compared to other products. The effect of moisture damage was also evaluated using AST test procedure. The effect of moisture reduced the maximum force for the reference and the reinforced specimen with grid A. The effect of moisture was found to be not significant for the specimens prepared with grids B, C and D.

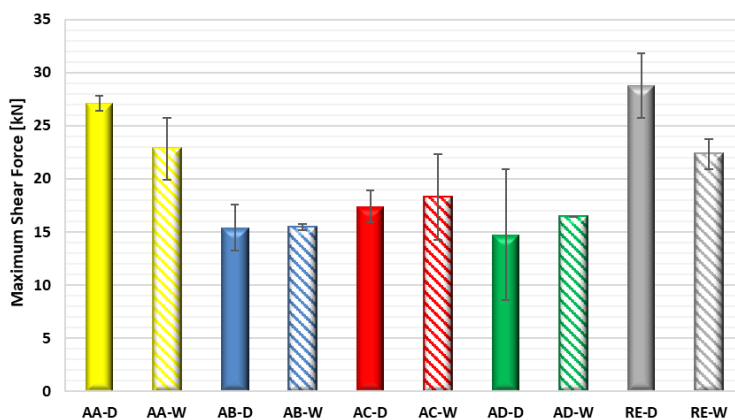


Figure 3: AST Maximum Shear Force, Comparison of Dry & Wet Tested Specimens (Figure 23 in report)

Interlayer bonding – in situ

The investigation on extracted field cores was conducted using the interlayer bond test (Leutner test). The test results presented in Figure 4 showed that the field cores of the unreinforced reference section had the highest interlayer bond values in comparison

to the reinforced sections irrespective of the reinforcement type used and therefore the best interlayer bond properties. On the other hand, the product used in Section A had the highest values followed by Section B, while Sections C and D never seem to be able to reach the standard requirement of 15 kN. From the results it is visible that the layer adhesion condition improved over time, a fact that is in accordance with earlier investigations (Raab et al., 2017, Raab & Partl, 2007). The increase of shear force for systems A, B and the reference are in the order of 40 %, while the increase for System C and D is 66 % and 76 % respectively. Although System A could proof its superior bonding qualities in the laboratory, it is worth noting that its installation was conducted early in the morning on already cured tack coat, while other systems (e.g. system B and D) were installed on a relatively uncured tack coat, which therefore affected the bonding quality. This is especially true for System B which uses a very high tack coat rate (dosage). Further, as highlighted by the producer of System C, the required installation temperature of the surface course of 140 °C, which is important to improve the bonding properties of the system, was not achieved.

Comparing the field results with the laboratory Leutner test the following can be said: Although only 1 or 2 field cores were tested for each reinforcement system, the ranking appears to be similar (System A best, followed by System B). Like in earlier investigations (Raab et al. 2017) there are systems that show in the laboratory a better or comparable behaviour to unreinforced specimens, while in in-situ, due to construction constrains and difficulties, the reinforcement always seems to provide a barrier between layers leading to lower interlayer bond properties. The poor performance of systems C and D was also found in the lab testing, while it is worth noting that System D tested on site (with chipping) was different to System D tested in the lab.

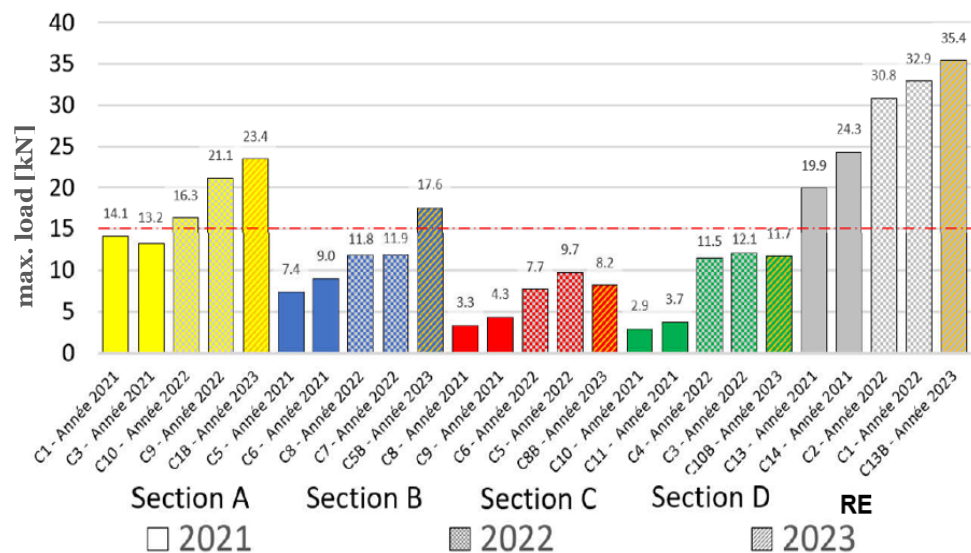


Figure 4: Field extracted cores Leutner results – Evolution 2021/2022 /2023 (Figure 73 in report)

Crack propagation MMLS3

The crack propagation tests conducted with the MMLS3 traffic load simulator revealed that System A outperforms all others, while System D performs poorly, carrying only about 60 % of the loads that System A can handle. Deformation sensor (LVDT) measurements showed that System A provides high stiffness to the structure, whereas System D is the weakest (see Figure 5).

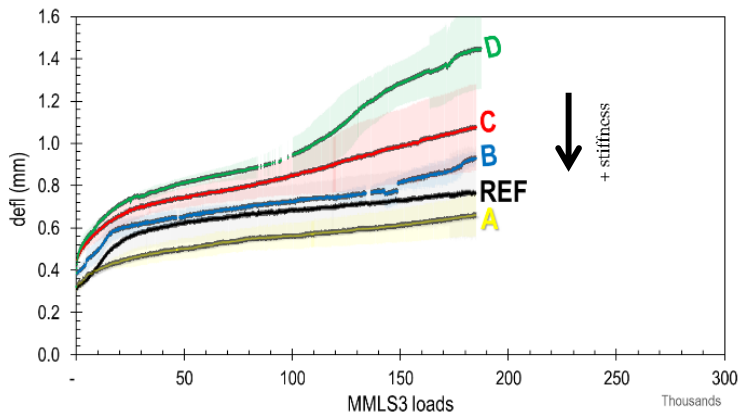


Figure 5: For each System Average deflection amplitude vs. no. of MMLS3 loads (Figure 57 in report)

Three distinct mechanisms of crack formation and propagation were identified, as shown in Figure 6 of the report:

1. For some specimens, immediately after the loading begins, the Digital Image Correlation (DIC) analysis shows debonding of the layers, followed by the initiation of a crack at the notch tip, travelling through the slab until reaching the upper surface. This mechanism was observed in all specimens of system D and two specimens of system C.
2. In the second mechanism, a vertical crack starts at the notch, propagates directly to the interface causing layer debonding, and then continues to the top. This suggests that the interlayer is slightly stronger than in the first mechanism. In both cases, the vertical crack can either continue directly in the upper layer or continue horizontally displaced. A horizontal displacement indicates a very weak interlayer, causing the layers to act independently under load. All specimens of system B, one specimen of system C and two Ref specimens followed this damage mechanism.
3. The third mechanism involves a vertical crack propagating directly to the top layer without causing interface separation. In the first two cases, crack propagation generally occurred faster than in slabs where no interlayer separation was observed. This was the cracking mechanism detected in all specimens of System A and one Ref specimen.

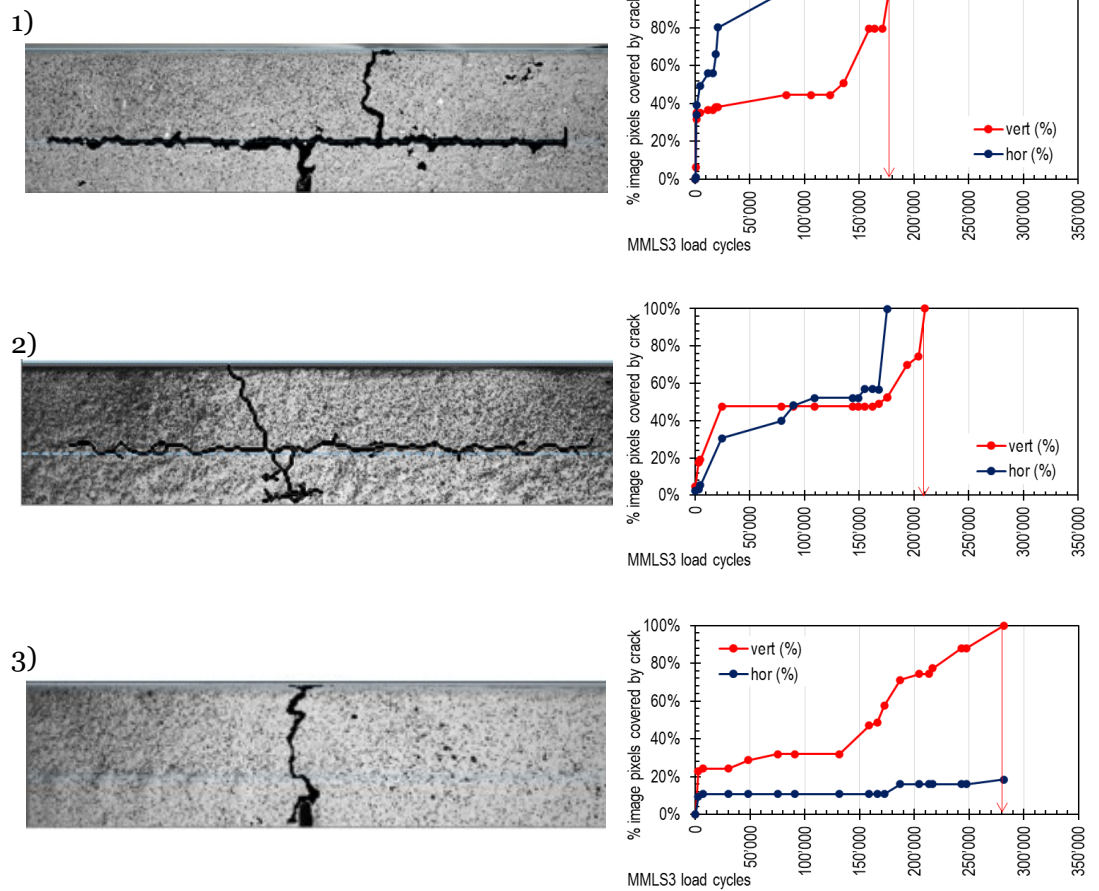


Figure 6: Three mechanisms of crack formation and propagation identified through the DIC analysis (Figure 63 in the report)

The summary of the crack propagation results in terms of MMLS3 loading cycles is presented in Figure 7.

The type of failure is strongly correlated with slab performance: slabs with interlayer debonding typically fail after fewer load cycles compared to those with intact interlayer bonds. In summary, the tests demonstrated that a grid system can effectively delay crack formation and propagation, provided it ensures strong bonding to the rest of the structure. This results in a stiffening effect for the entire system. Consequently, all reinforcement systems, with the exception of System D, outperformed the unreinforced reference.

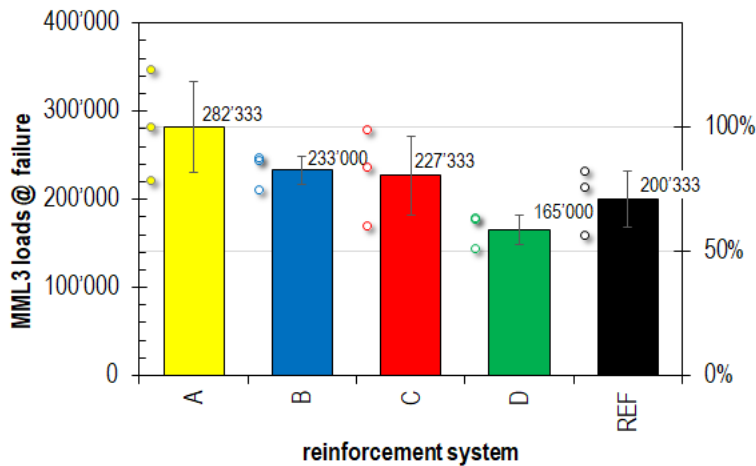


Figure 7: Approximate number of loads until failure of the slabs (Figure 64 in report)

Crack propagation – 4-Point Bending

Beam specimens were taken from the double layer slabs for the investigation of the crack propagation test. The 4-point bending (4PB) beam crack propagation test results shown in Figure 8 revealed that all reinforced specimens sustained a higher number cyclic loading irrespective of the grid type in comparison to the unreinforced reference specimens. The specimens with grid reinforcement B dominated and offered the highest resistance against crack propagation followed by product D.

To evaluate the effects of reinforcement joints, as they appear on site due to construction (e.g. reinforcement grid joints along the pavement length i.e. longitudinal direction and across the pavement width i.e. transversal direction), the overlap lengths test was conducted on specimens prepared for the low temperature crack propagation test. The situation of the overlap length of the grid reinforcement, more precisely in the longitudinal direction along the road, was assumed in the samples prepared in the laboratory and subjected to crack propagation tests under cyclic loading. All specimens were prepared with an overlap length of 15 cm of the reinforcement grid.

The test results revealed that the specimens prepared with grid reinforcement using System B sustained a higher number of cycles against crack propagation and even showed higher resistance compared to the full-length grid. These results indicate that the overlap length offers higher resistance using the crack propagation test. This effect was visible for all the reinforced specimens except for product D. Where, the ranking order as evidenced in the results of specimens prepared with full length grids improved after testing the specimen in overlap length situation. It is worth to mention that the number of cycles for product D specimen with overlap length reached even lower number of cycles in comparison to the reference specimens. This could be related to the interface situation of specimen with product D, as in the results of AST test, it was experienced that the specimens prepared with grid product D had problems with the interface in dry and wet situation. Furthermore, product D received the lowest values in the AST.

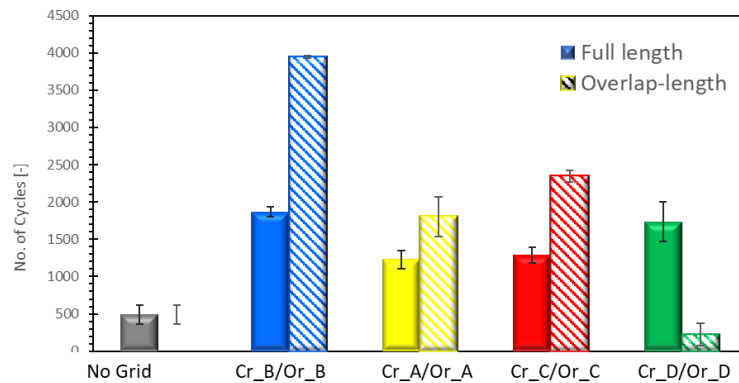


Figure 8: Comparison of reinforced (full and overlap length) grid samples in terms of total number of cycles (Figure 41 in report)

Test section

The construction of a test track T9 Sierre located at Route d'Escala, chainage PR580+740m to PR590+040m in Sierre (canton of Valais) was carried out on 15.10.2021. In parallel to the laboratory investigation, the field test track preparations were started in coordination with the Cantonal authorities and the producer of the mixtures. All grid reinforcement suppliers were invited to the test track construction to lay down their grids according to their defined procedures. Five different test section namely, Section A, Section B, Section C, Section D (with grid reinforcement) and a reference Section without grid reinforcement were built. Condition surveys were performed before and after the construction of the test track in yearly intervals. Each condition survey included the extraction of cores for layer adhesion Leutner test, Falling weight deflectometer (FWD), ARAN survey and in some cases a visual inspection. The condition indices I_1 , I_2 and I_3 shown in Table 2, Table 3 and Table 4, as well as the macrotexture values were recorded using the ARAN measuring vehicle. The surface deterioration index I_1 showed for all test sections both reinforced and unreinforced a mix of good and average conditions according to the Swiss standard VSS40925. The longitudinal evenness index I_2 results for the reference section showed critical condition along with a mix of good and average conditions. For Section B a small part in one direction was identified to be in sufficient condition, while the rest of the Section showed a mix of good and average conditions. The transversal evenness index I_3 showed that the reference Section and Section B were mostly in good condition, while Sections A, C and D, that were found mostly in average condition. Falling Weight Deflectometer (FWD) measurements were performed, and the results were backcalculated to a reference temperature of 15 °C. The backcalculated elastic moduli of the asphalt concrete layer, as shown in Figure 9, indicated that the unreinforced section exhibited slightly higher values compared to the reinforced sections. This difference may be attributed to the presence of reinforcement at the interface, which could affect the stress distribution. Among all the sections with reinforcement grids, the asphalt layer with Product A demonstrated slightly higher moduli values than the asphalt layers with other reinforcement systems.

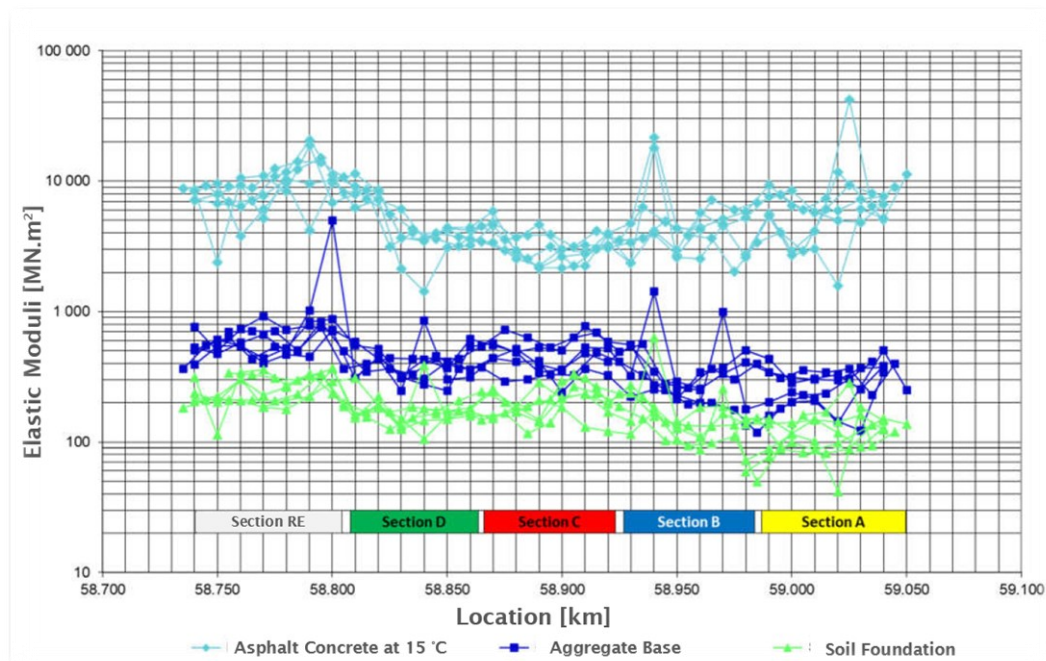


Figure 9: Back calculated elastic moduli of test section (Figure 80 in report)

Surface degradation Index I_1

Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Average	0.9	1.1	1.4	1.2	1.4	1.1	0.9	1.3	1.3	1.1
Minimum	0.8	0.8	1.4	0.8	1.4	0.8	0.8	0.8	0.8	0.8
Maximum	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

Table 2: Details results of surface degradation Index I_1 (Table A8 in Annex)

Longitudinal Index I_2

Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Average	1.1	1.0	1.0	0.5	0.8	0.9	2.2	1.4	1.0	0.7
Minimum	0.3	0.4	0.5	0.0	0.3	0.2	1.5	0.2	0.6	0.4
Maximum	1.6	1.4	1.5	1.0	1.8	2.1	3.2	2.2	1.5	1.0

Table 3: Details results of longitudinal Index I_2 (Table A9 in Annex)

Transverse Index I_3

Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Average	0.8	1.0	1.4	1.2	1.3	1.2	0.6	1.2	1.3	0.9
Minimum	0.5	0.7	1.0	0.6	1.1	0.8	0.3	0.9	1.0	0.3
Maximum	1.1	1.4	1.6	1.5	1.5	1.7	1.3	1.5	1.5	1.5

Table 4: Details results of transverse Index I_3 (Table A11 in Annex)

Moreover, the surface condition regarding crack appearance was evaluated during the measurement campaign in 2023. The evaluation showed that the cracks in the reference section were mostly in transverse direction, while the cracks in the reinforced sections were mainly in longitudinal direction. Overall, the number of visible cracks was higher in the reference section compared to the reinforced sections. For a more accurate crack evaluation the ARAN data were further processed using imaging technique. In this evaluation, the number of cracks was compared and evaluated based on the number of cracks before the construction of the test track. Note that the comparison was made based on the available pre-construction data in 2019 in one direction only (i.e. towards Sierre). The columns in Figure 10 show the number of pixels based on the ARAN measurements in 2019 before and in 2023, two years after the construction of the test track. As can be seen from Figure 10, the number of cracks in 2019 (based on the pixel count) is much higher in the reference section compared to all reinforced sections. However, the measurements in 2023 show that the percentage of cracks after 2 years is still higher in the reference section in comparison to the reinforced sections.

Figure 11 shows the number of cracks in 2023 as a percentage of the number of cracks in 2019 for all sections. As mentioned earlier the highest percentage with ca. 61% can be found for the reference section, followed by Section B with a percentage of 50 %. Section A has the lowest number of cracks relative to the number of cracks before construction (ca. 9 %), while the percentage for Section C is ca. 25 % and 33% for Section D. The percentage of 9 % for Section A does not include a construction joint that re-appeared on the surface and therefore is not related to grid performance.

Since for the performance evaluation of the grids the number of reflective cracks is relevant, the percentage of reflective cracks (comparing the number of cracks before and after construction) for each section was evaluated as shown in Figure 12. Again, the reference section without reinforcement has the highest number of reflective cracks (43 %) followed by Section B with ca. 20 %. In Section A, the number of reflective cracks is ca. 2 % (excluding the construction joint, which is not considered a reflective crack), while in Section C and D no reflective cracks can be found.

The investigations revealed that the installed reinforcements are quite capable of preventing or at least delaying the appearance of reflective cracks.

However, it is difficult to compare the reinforcement grids installed in the different sections with each other, as an inspection in May 2024 highlighted very different geometries in the form of curves and junctions and thus different traffic loads for the individual sections. In this context, the roundabout presence at the beginning of Section A and turn-off lane in Section B must be mentioned.

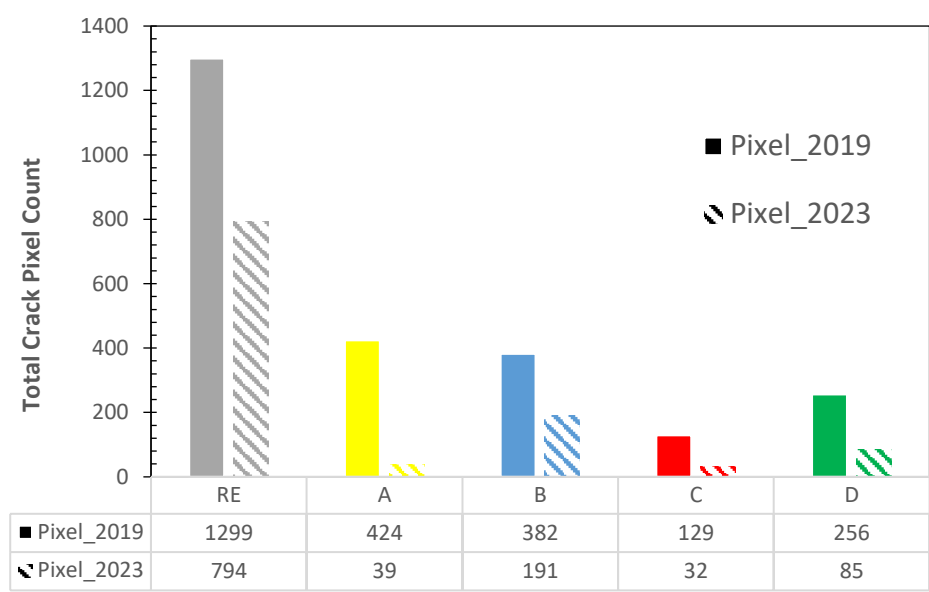


Figure 10: Surface crack pixel value measured using imaging technique (Figure 88 in report)

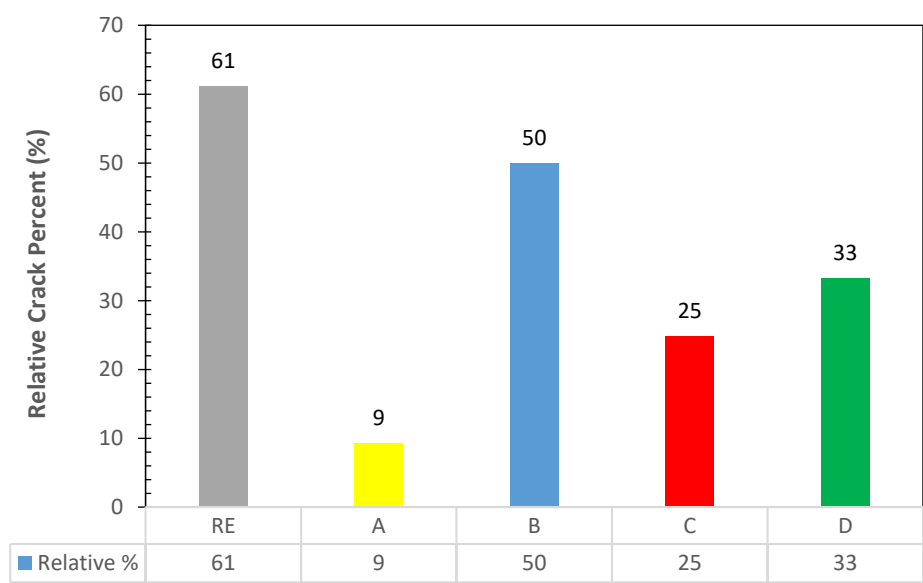


Figure 11: Surface crack pixel value in relative percentage (Figure 89 in report)

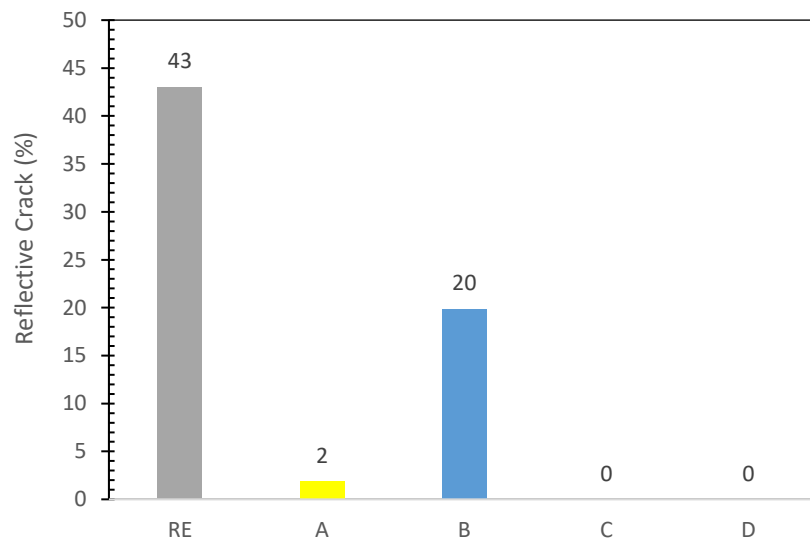


Figure 12: Reflective surface crack pixel value in relative percentage (Figure 90 in report)

Modelling

The modelling efforts comprised the Leutner and the MMLS3 test, with the aim of establishing the basis for taking reinforcement grids into account in the pavement dimensioning process. In order to do this, the finite element method (FEM) was chosen as the most suitable tool for modelling the complex geometry of the grids and the interaction between them and the surrounding structure. The finite element model used in this project was based around the commercial, Abaqus code. In a first stage, a model of the Leutner test was prepared using Abaqus/CAE environment in order to establish the most appropriate interlayer bonding model and their parameters. After trying different interlayer bond models for the interlayer, cohesive elements were selected to simulate the interaction between asphalt layers. The left side of Figure 13 shows the geometry of the model and on the right, the comparison of the model results with the measurement is presented. If the parameters are correctly chosen, the model can accurately predict the test.

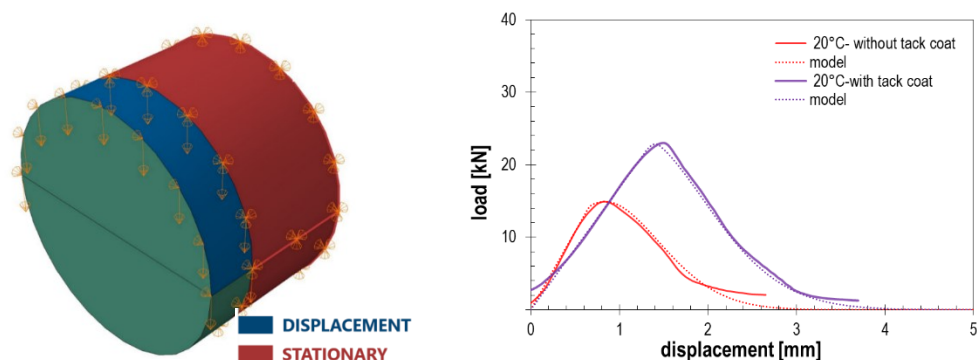


Figure 13: FEM geometry of the Leutner test (left) and comparison of the load displacement curves obtained with the test and the model (Figure 98 in report)

Simulations of the MMLS3 test were prepared using an Abaqus plug-in programmed in Python. The plug-in shows a Guided User Interface (GUI) form in the CAE environment from Abaqus (Finite Element Software). In this form, the user can enter all the parameters of the simulations, including the geometry of the slabs and the grids, the materials, the load, the interaction properties between layers and the meshing parameters. With this procedure, it is possible to create different advanced 3D simulations using moving loads, temperature dependent viscoelastic asphalt materials, embedded reinforcement grids of various geometries and different interlayer bonding parameters, including the cohesive elements with parameters calibrated from the previous Leutner models (see Figure 14). The model was calibrated using the measured displacements to achieve a good fit, as shown in Figure 15 and results comparing the strains for the not reinforced and the grid reinforced situation showed a slight difference (see Figure 16). Starting from this point, the model can be extrapolated to a real pavement model and different reinforcement grids can be evaluated without the need of testing.

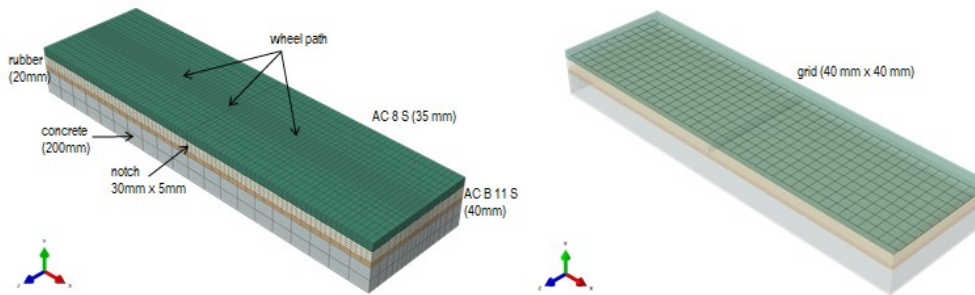


Figure 14: View of the geometry and mesh of the model (Figure 110 in report)

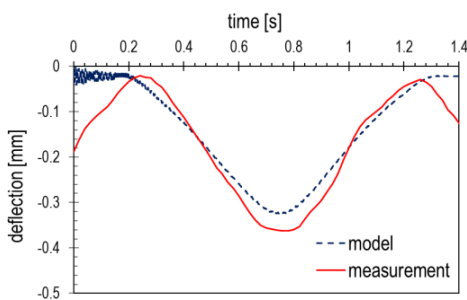


Figure 15: Calibration of the model (Figure 114 in report)

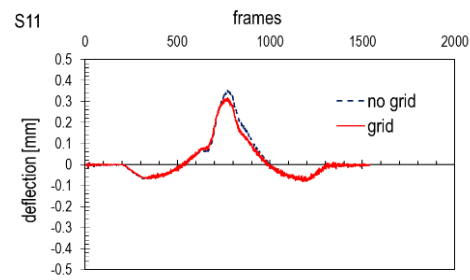


Figure 16: Strains in the longitudinal direction for an element on top of the notch, in a model with and without grid (Figure 116 in report)

The validation results confirm the accuracy of the developed model, providing a basis for further analysis incorporating specific grid properties, such as rib spacing, material modulus, and cross-sectional area. Future work should include Leutner tests at various temperatures to refine interlayer bonding parameters and cohesive element values. The model can then evaluate the effects of grid placement and conditions like load, speed, and temperature on pavement stress and strain. Expanding simulations to full-scale pavements under traffic loads will generate critical data for designing reinforced pavements using mechanistic-empirical approaches.

LCCA

Based on the results of the previous working packages a life-cycle cost analysis (LCCA) was applied. The basis for the LCCA is a theoretical service life model based on key performance indicators, which was adapted for three different scenarios with regards to test results available from the project. The MMLS3 load tests were considered as relevant due to their realistic test setup. The initial situation was based on an existing asphalt road section visibly damaged by cracks on the road surface course. The asphalt layers below have already reached most of their theoretically expected service life. The key question was whether a full reconstruction (replacement of all asphalt layers) or a more targeted rehabilitation approach—replacing only the asphalt surface layer combined with the use of asphalt reinforcement—would be more effective. To evaluate this, three different scenarios were selected and their performance compared using defined key performance indicators (KPIs). The reference variant Ref includes a replacement of all asphalt layers at the beginning of the analysis period. Variants V1 and V2 consisted of replacing the asphalt surface layer while incorporating asphalt reinforcement between surface and binder layer. This approach aims to delay the need for a full replacement of all three asphalt layers. These two variants V1 and V2, derived from the results of the MMLS3 load tests, reflect the range of existing pavement quality in combination with the asphalt reinforcement system used.

Variant V1 assumes that optimum conditions prevailed during paving, leading to the best possible paving quality. This means that the interlayer bond between the grid and asphalt layers could be optimally achieved. Variant V2, on the other hand, assumes that an optimum bond between the asphalt grid and the asphalt layers could not be achieved. As a rule, this means that the traffic loads applied to the asphalt surface courses cannot be optimally transferred to the underlying layers and the asphalt surface layer fatigues prematurely.

The benefit of each variant was calculated for each KPI and quantified using the sum of the areas under the KPI curves. The corresponding costs were limited to the measure-related costs. The three model variants show similar benefits in terms of the performance of the four KPIs. The reference variant Ref achieved a total area of 869.5 units, Variant V1 a total area of 867.9 units, and Variant V2 a total area of 866.6 units. However, there is a significant difference between the three model variants in terms of total costs. Variant V2, with total costs of CHF 208,687, is the least favourable option, as it requires an additional full reconstruction (E2) within the analysis period. In contrast, the reference variant Ref with CHF 165,672 and variant V1 with CHF 147,965 delivered lower costs.

The results of this use case show that the use of asphalt reinforcement can provide an additional benefit regarding a long-term analysis period of 50 years. If the asphalt reinforcement is not installed optimally or cannot fulfil the minimum requirements, such as layer bonding, an additional maintenance measure may be necessary, leading to increased costs within the life cycle. However, the use of asphalt reinforcement can have an advantage where the technically appropriate replacement of all asphalt layers is temporarily not feasible due to traffic, economic, personnel or other reasons. In addition, the impact of the necessary dumping of asphalt mill material contaminated with reinforcement mesh was investigated. Depending on the product-specific Environmental Product Declarations (EPDs) used to assess CO₂ equivalent units and other sustainability indicators, this can lead to further decrease in benefit.

Discussion

Based on the results obtained from this project the following discussion points are made:

- The interlayer bond test shows that there are reinforcement grids which in the laboratory lead to better or comparable behaviour compared with unreinforced specimens, while in situ, due to construction constraints and difficulties, the reinforcement grids always provide a discontinuity between layers, often leading to lower interlayer bond properties.
- The in-situ interlayer bond generally improves over time, a fact that is in accordance with earlier investigations (Raab et al., 2017, Raab & Partl, 2007). Some reinforcement suppliers even claim this mechanism (compacting traffic load in combination with high temperatures in summer) to improve the performance of their product.
- The comparison between MMLS3 and 4PB crack propagation tests was found to be not straight forward. This could be related to the laboratory production of specimens and the different scales of specimens used for each type of test. Furthermore, the test temperature was a determining factor as the MMLS3 specimens were tested at 20 °C, while the 4PB test was conducted at 5 °C.
- Overall, the MMLS3 test must be considered more realistic and comparable to the field situation due to its load configuration and the size and geometry of the specimens, especially regarding the mesh size of the reinforcement products. Furthermore, it is worth mentioning that the 4PB crack propagation test was carried out testing two specimens only.
- The specimens with an overlap length of 15 cm were found to be more resistant in the 4PB test, compared to the full-length specimens in the similar test. This shows that

overlapping of the tested reinforcement grids according to the suppliers' recommendations does not provide a problem and that the overall overlap length for a specific reinforcement product could even be optimized.

- The selected test track was part of the scheduled maintenance work of Canton of Valais and was managed in collaboration with the Cantonal authorities. Although it was chosen due to its high amount of heavy traffic, neither the geometry (curves), the amount and regime of traffic (amount of heavy trucks, turning lanes, shear forces due to breaking at roundabouts) nor the damage (number of cracks) prior to the rehabilitation was identical for all sections. As visible in an inspection in 2024 sections with aggregate loss and raveling of the surface layer were found, phenomena that cannot be attributed to the performance of the reinforcement but reveal the different loading stresses of the sections.
- Regarding the installation of the different reinforcement grids, it is worth noting that the installation of System A was conducted early in the morning on already cured tack coat, while other systems (e.g. system B and D) were installed on a relatively uncured tack coat. This is especially true for system B with a very high tack coat rate. Further, as highlighted by the producer of a system with self-adhesive membrane, the optimal temperature of the surface layer of 140°C, which they consider important to ensure the bonding properties of their system, was not achieved.
- A comparison between laboratory and field results was not possible for reinforcement product D since chipping was used for the field installation but not in the lab.

Conclusions and recommendations

- As mentioned above, the interlayer bond test revealed that there are reinforcement grids, which in the laboratory lead to better or comparable behavior compared with unreinforced specimens, while in situ, due to construction constraints and difficulties, the reinforcement grids always seem to provide a discontinuity between layers often leading to lower interlayer bond properties. Nevertheless, there are reinforcement systems that can achieve the required average value of 15 kN according to the Swiss standard (SN-EN-12697-48). For this reason, it is very important that manufacturers continue to improve their systems regarding layer bonding on the one hand and optimize the installation of their products as far as possible on the other.
- For their part, road owners and contractors must ensure that the necessary requirements for pavement construction are strictly met when installing reinforcement systems and that all conditions for an optimal installation of specific reinforcement grid are fulfilled.
- By using different laboratory test methods to determine the crack propagation (MMLS3 and 4PB), the crack-bridging and crack-retarding effect of asphalt reinforcements grids and the differences to systems without asphalt reinforcement could be clearly demonstrated for various asphalt reinforcement systems. Also in this respect, the interlayer bonding was found to be one of the controlling factors for the performance of a reinforced pavement system. To achieve better performance, it is therefore very important to improve interlayer bonding, which is also crucial for preventing moisture damage.

- It was demonstrated that the combination of laboratory-scaled traffic simulator tests (MMLS tests), deflection measurements with LVDTs and digital image correlation (DIC) data analysis is an important tool to understand the mechanism of reinforcement systems, especially with the newly proposed way of determining horizontal and vertical crack propagation patterns. The evaluation of horizontal and vertical crack propagation demonstrates that the performance of reinforcements depends on different mechanisms and that it may be misleading to rely only on small-scale tests.
- The results of overlap length testing showed that overlapping of the tested reinforcement grids, if done according to the suppliers' recommendations, does not provide a problem and that the overlap length for a specific reinforcement product could even be optimized.
- When looking at the performed testing, as mentioned above, the MMLS3 crack propagation test must be considered more realistic and comparable to the field situation compared to the 4PB test due to its load configuration and geometry of the specimens, especially regarding the mesh size of the reinforcement products. Although the 4PB crack propagation test has its limitations, it is easier to perform and useful for comparing different products. To increase the confidence level of the test results, it is essential to conduct it using a larger number of specimens (not only 2 specimens).
- An accurate numerical simulation of MMLS3 tests was developed and validated, demonstrating high accuracy in predicting structural responses. The model incorporates advanced material properties, loading conditions, interlayer bonding, and grid geometry. It distinguishes the behavior of slabs with and without reinforcement, with future work needed to evaluate individual grid types.
- Extending this modeling will enable the evaluation of reinforcement systems using supplier data, reducing the need for extensive physical testing and allowing easy assessment under varying conditions. This provides critical inputs for mechanistic-empirical pavement design, ensuring more accurate and efficient dimensioning of reinforced pavements.
- When looking at the test track, visual surface condition evaluations showed that the number cracks was higher in the reference section compared to the reinforced sections. The cracks in the reference section were mostly in transversal, while the cracks in the reinforced sections were mainly in longitudinal direction.
- For a more accurate crack evaluation the ARAN data were processed using imaging technique. According to this evaluation (in direction Sierre) the number of cracks was compared and evaluated based on the number of cracks before the construction of the test track. Here it was found that the number of cracks before construction had been already much higher in the reference section compared to all reinforced sections. However, when calculating the percentage of cracks before and 3 years after construction, the number of cracks in the reference section was still higher than the number of cracks in the reinforced sections.
- For the reference section the highest percentage with ca. 61% was found, followed by Section B with a percentage of 50 %. Section A had the lowest number of cracks relative to the number of cracks before construction (ca. 9 %), while the percentage for sections C and D was 25 % and 33% respectively. Furthermore, the percentage of 9 % for Section A does not include a construction joint that re-appeared on the surface and therefore could not be related to grid performance.

- Since for the overall performance evaluation of the grids, the number of reflective cracks is relevant, the percentage of reflective cracks (comparing the number of cracks before and after construction) for each section was evaluated. Again, the reference section showed the highest number of reflective cracks (43%) followed by Section B with 20 %. In Section A, the number of reflective cracks was 2 %, while for sections C and D no reflective cracks could be found.
- The field investigations therefore revealed that the installed reinforcements are quite capable of preventing or at least delaying the appearance of reflective cracks.
- However, it was difficult to compare the reinforcement grids installed in the different sections with each other, as an inspection in May 2024 confirmed very different geometries in the form of curves and junctions and thus different traffic loads for the individual sections. In this context, the roundabout presence at the beginning of Section A and turn-off lane in Section B must be mentioned.
- All evaluations and tests performed during the project revealed that the number of laboratory and field specimens must be increased to be able to present reliable findings and provide recommendations for standardization and application. The modelling developed in the frame of this research project will allow for determining the performance of different reinforcement systems using product data provided by the reinforcement suppliers. Using this tool the amount of testing could be reduced significantly, and different testing conditions could easily be evaluated.

Zusammenfassung

Einleitung

Asphaltbeläge, die starkem Verkehr und Witterungseinflüssen ausgesetzt sind, stossen mit der Zeit an ihre Tragfähigkeitsgrenzen und weisen verschiedene Arten von Schäden (Risse, Spurrinnen usw.) auf, die die Verkehrssicherheit und den Fahrkomfort der Verkehrsteilnehmenden beeinträchtigen. Daher ist es wichtig, effiziente Instandsetzungs- und Erneuerungsmethoden zu suchen, die es ermöglichen, die Erhaltungskosten zu senken, die Lebensdauer zu verlängern sowie Material und Energie einzusparen. Für die Erhaltung der Strasseninfrastruktur stehen vielfältige Massnahmentypen zur Verfügung, wobei sich Erhaltungsmassnahmen mit dem Einsatz von Asphaltbewehrungen vor allem zur Vermeidung von Reflektionsrissen immer mehr durchsetzen. Auch die internationale Literatur zeigt, dass sich Asphaltbewehrungen im Allgemeinen positiv auf das Zustandsverhalten (Performance) von Asphaltstrassen auswirken. Es gibt jedoch noch viele offene Fragen, insbesondere hinsichtlich ihrer Auswirkungen auf den Schichtenverbund. Darüber hinaus lieferte das ASTRA-Forschungsprojekt 2011/O11 sehr interessante Ergebnisse zum Einsatz von Asphaltbewehrung und dem damit verbundenen Performanceverhalten (Festigkeit und Dauerhaftigkeit). Die Hersteller von Asphaltbewehrungsgittern haben ihrerseits ein gewisses Know-how entwickelt, das es ihnen ermöglicht, ihre Produkte erfolgreich auf Baustellen einzusetzen. Alle oben erwähnten Untersuchungen basieren jedoch auf unterschiedlichen Methoden und Prüfverfahren, und die dabei erzielten Ergebnisse sind grundsätzlich nur für das jeweilige Produkt gültig.

Auch wenn sich die Wirkung von Asphaltbewehrungen in vielen Fällen in der Praxis bewährt hat, gibt es derzeit keine verlässliche Theorie, um die Mechanismen und die Wirksamkeit ihres Einsatzes zu beschreiben. Daher besteht der Bedarf an aussagekräftigen Untersuchungs- und Prüfmethoden, die einen direkten Vergleich verschiedener Bewehrungsgitter sowie deren Gesamtwirksamkeit und Einfluss auf die Performance des Asphaltaufbaus ermöglichen.

Um diese Lücke zu schliessen, wurde im aktuellen Forschungsprojekt die Leistungsfähigkeit (Performance) unterschiedlicher Bewehrungsgitter anhand verschiedener Laborprüfungen auf der Grundlage unterschiedlicher Probengrössen und -abmessungen bewertet, um Erkenntnisse über den Einsatz von Bewehrungsgittern zu gewinnen. Parallel dazu wurde eine Teststrecke gebaut, um das in-situ-Verhalten der Asphaltbewehrungen zu überwachen und mit den Ergebnissen der Laboruntersuchungen zu verknüpfen.

Ziele

Das Projekt zielte darauf ab, bewährte Verfahren für den Einsatz von Bewehrungsgittern in Asphaltaufbauten zu untersuchen und zu fördern, indem die methodische und normative Grundlage geschaffen wurde, die erforderlich ist, um solche Bewehrungsgitter beim Dimensionierungsprozess zu berücksichtigen und ihre Wirksamkeit (Performance) zu bewerten. Die Bewertung erfolgte hierbei hersteller- und produktunabhängig, weshalb die wichtigsten in der Schweiz tätigen Hersteller von Asphaltbewehrungsgittern berücksichtigt wurden. Alle Ergebnisse dieser Arbeit werden jedoch anonym, d.h. ohne Angabe des Herstellers dargestellt.

Methodologie

Die in diesem Projekt angewandten Methoden basieren auf Labor- und in-situ-Untersuchungen sowie einer Lebenszyklusanalyse. Die Laboruntersuchungen wurden an der BFH und der Empa durchgeführt. Um die benötigten Proben für die Laboruntersuchungen an der BFH herzustellen, wurden die herstellerepezifischen Produktinformationen dokumentiert und anschliessend im Labor der BFH eine praktische Demonstration der Einbaumethoden der jeweiligen Hersteller der Asphaltbewehrungsgitter durchgeführt. Anschliessend wurden die Probekörper nach den gegebenen Anweisungen hergestellt. Da es sich bei den an der Empa geprüften Proben um eine grössere Skalierung handelte, wurden die Hersteller aufgefordert, ihre Produkte selbst einzubauen, d.h. Vertreter der jeweiligen Lieferanten sollten das für den Einbau benötigte Material zur Verfügung stellen und den Einbau selbst durchführen. Die gesamte Probenvorbereitung im Labor erfolgte nach den von den Asphaltlieferanten festgelegten Anforderungen und Richtlinien und unter Verwendung der von ihnen bereitgestellten Materialien.

Die verschiedenen Phasen des Projekts sind in Abbildung 1 dargestellt. Nach einer Aktualisierung des Wissensstandes und dem Studium der neueren nationalen und internationalen Literatur (Task 1) wurde eine experimentelle Untersuchung im Labor durchgeführt (Task 2). Diese Untersuchungen umfassten u.a. die Bewertung des Schichtenverbunds (Adhäsion/Verzahnung zwischen den Schichten), die Untersuchung der Ermüdungsfestigkeit und der Rissbildung (4-Punkt-Biege- und Verkehrslastsimulator-Versuche) sowie die Bewertung der Überlappungslänge. In den in-situ-Untersuchungen (Task 3) wurden die praktischen Aspekte des Einbaus von Asphaltbewehrungsgittern und die Überwachung ihrer in-situ Performance und somit das Verhalten unter realer Beanspruchung (Verkehr und Klima) analysiert. Zu diesem Zweck wurden regelmässige Messungen mit dem Automatic Road Analyzer (ARAN) und dem Falling Weight Deflectometer (FWD) durchgeführt und Bohrkerne entnommen. Aufgrund der im Labor und in-situ-ermittelten Ergebnisse wurde eine FEM-Analyse von bewehrten Asphaltaufbauten (Task 4) und zusätzlich eine Lebenszykluskostenanalyse (LCCA) durchgeführt. Alle ermittelten Daten und gewonnenen Erkenntnisse wurden genutzt, um detaillierte Empfehlungen für die Planung und Durchführung von Erhaltungsmaßnahmen mit Asphaltbewehrungsgittern unter Berücksichtigung der In-situ-Bedingungen (klimatische Bedingungen, Zustand der aufnehmenden Schicht, Installationsbedingungen usw.) zu erarbeiten (Task 6).

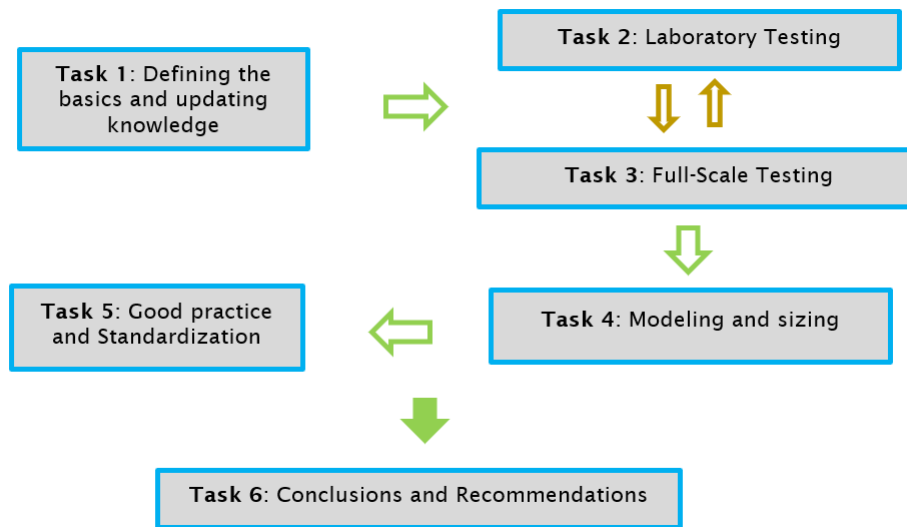


Abbildung 1: Projektphasen (Abbildung 6 im Bericht)

Materialien

Für alle in diesem Projekt durchgeführten Prüfungen und Untersuchungen wurden Asphaltbetonmischungen der Spezifikation AC 8 S (Deckschichten) und AC B 11 S (Binderschichten) verwendet. Die Mischungen wurden aus einem Asphaltwerk in Hinwil bezogen. Der verwendete Haftkleber war eine polymermodifizierte, kationische Emulsion mit einem Restbitumengehalt von 60 %. System C benötigte aufgrund seiner selbstklebenden Membran keinen Haftkleber.

Es wurden vier verschiedene Arten von Bewehrungsgittern ausgewählt. Die Zusammenfassung der wichtigsten mechanischen Eigenschaften dieser Bewehrungsgitter ist in Tabelle 1 dargestellt. Um die Anonymität der Ergebnisse zu gewährleisten, sind die Produkte mit den Buchstaben A, B, C und D gekennzeichnet.

Bewehrungsgitter

Produkt	A		B		C		D	
Haftkleber Teststrecke (residual) [g/m ²]	180		690		keiner*		420	
Haftkleber Labor (residual) [g/m ²]	180		480		keiner		250	
Richtung der Litze	L	T	L	T	L	T	L	T
Material des Strangs	Glas	Kohlen- stoff	Polyester		Glas		Glas	Kohlen- stoff
Maschen- weite [mm]	15	15	40		10		18	18
Zugfestigkeit [N/m]	120	200	50		100		120	200
Dehnung [%]	3	1.5	12		3		3	1.5

*Der Haftkleber wurde nur an den Enden aufgebracht (600g/m²)

Tabelle 1: Mechanische Eigenschaften des im Projekt verwendeten Bewehrungsgitters (Tabelle 2 im Bericht)

Ergebnisse

Schichtverbund - Labor

Die Laboruntersuchung umfasste die Herstellung von Probekörpern in verschiedenen Grössen. An der Empa und der BFH wurden zweischichtige Belagsplatten für die Beurteilung der Schichthaftung und der Rissausbreitung hergestellt. Daraus konnten Bohrkerne für die Prüfung des Schichtenverbunds mittels Leutner- und Advance Shear Test (AST) entnommen werden.

Die Auswertung des Schichtenverbunds mittels Scherprüfung nach Leutner zeigte, dass alle Asphaltbewehrungsgitter, mit Ausnahme des Systems A, einen negativen Einfluss auf den Verbund haben. In der Laboruntersuchung unterschieden sich die maximalen Verbundkräfte der untersuchten Bewehrungssysteme dabei recht deutlich. Insbesondere System A erreichte mit durchschnittlich 33,2 kN sehr hohe Werte, gefolgt von System B mit einem Durchschnittswert von knapp 25,8 kN, während System C mit einer durchschnittlichen max. Verbundkraft von weniger als 6,0 kN sehr schlecht abschneidet. System D erreichte einen Mittelwert von 16,0 kN, aber unter Berücksichtigung der Standardabweichung konnte, der nach Schweizer Norm SN EN 12697-48 geforderte Wert von 15 kN nicht für alle getesteten Proben erreicht werden. System A erhielt sogar höhere Werte als das unbewehrte System (Referenz), welches eine durchschnittliche max. Verbundkraft von 30,3 kN erreicht. Die Ergebnisse sind in Abbildung 2 dargestellt.

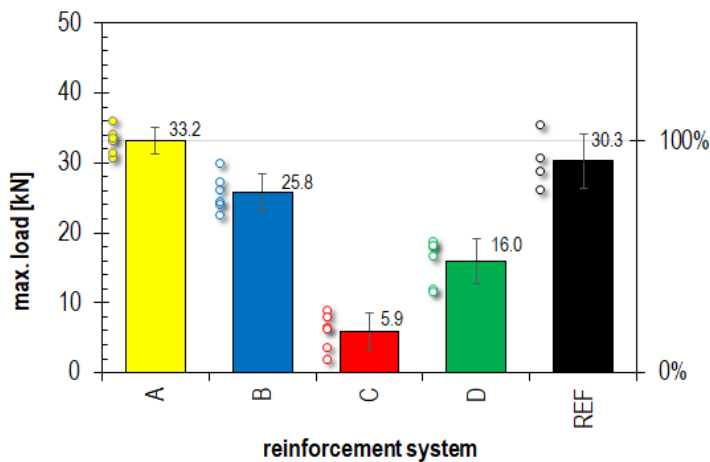


Abbildung 2: Ergebnisse des Leutner-Prüflabors für jedes Verstärkungssystem (Abbildung 15 im Bericht)

Der Schichtenverbund der Referenzprobe (REF), d.h. ohne Asphaltbewehrung, erreichte bei den Laborprüfungen mittels AST-Test höhere Werte im Vergleich zu den Probekörpern mit Asphaltbewehrung. Die Ergebnisse in Abbildung 3 (farbige Balken, D-dry engl. für trocken) zeigen für das Produkt A (AA) im Vergleich zu den anderen Produkten B (AB), C (AC) und D (AD) die höchste Maximalkraft auf. Auch nach der Wasserlagerung hat das Produkt A die höchsten Verbundwerte im Vergleich mit den anderen Bewehrungsgittern. Die Auswirkung von Feuchtigkeit auf den Verbund wurde ebenfalls mit dem AST-Testverfahren bewertet (vgl. Abb. 3, schraffierte Balken, W-wet engl. für nass), die Wirkung konnte aber bei den Probekörpern nicht durchgängig als negativ festgestellt werden. Durch die Einwirkung von Feuchtigkeit verringerte sich die maximale Kraft der Referenz und der Proben mit Gitter A. Ein Einfluss der Feuchtigkeit auf den Schichtenverbund bei den Proben mit den Bewehrungsgittern B, C und D konnte nicht eindeutig festgestellt werden.

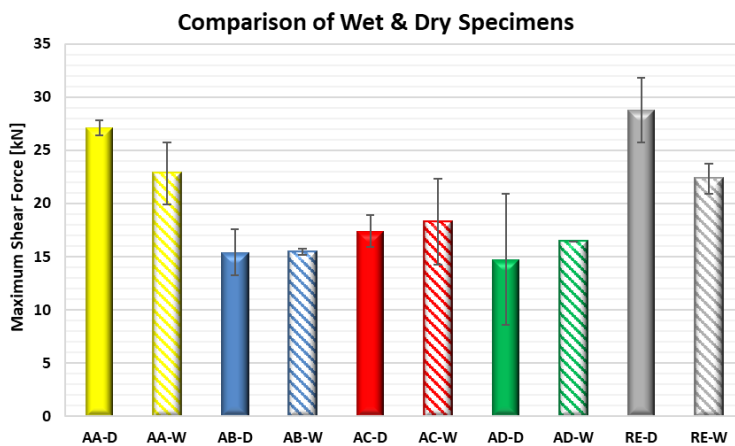


Abbildung 3: AST Maximale Scherkraft, Vergleich von trocken und nass geprüften Proben (Abbildung 23 im Bericht)

Schichtenverbund – in situ

Die Untersuchung des Schichtenverbunds wurde mit der Scherprüfung nach Leutner an entnommenen Bohrkernen aus der Versuchstrecke durchgeführt. Die in Abbildung 4 dargestellten Versuchsergebnisse zeigten, dass die Bohrkern des unbewehrten Referenzabschnitts im Vergleich mit den Abschnitten mit Bewehrungsgittern - unabhängig von der Art des Bewehrungsgitters - die höchsten Scherkräfte und damit den besten Schichtenverbund aufwiesen. Unter den Bewehrungsgittern wies das in Abschnitt A verwendete Produkt die höchsten Werte auf, gefolgt von Abschnitt B, während die Proben aus Abschnitten C und D weit unter der Normanforderung von 15 kN blieben. Aus den Ergebnissen geht hervor, dass sich der Schichtenverbund im Laufe der Zeit verbesserte, was mit früheren Untersuchungen übereinstimmt (Raab et al., 2017, Raab & Partl, 2007). Die Zunahme der Verbundkraft für die Systeme A, B und die Referenz liegt in der Größenordnung von 40 %, während die Zunahme für die Systeme C und D 66 % bzw. 76 % beträgt.

Obwohl System A seine überragenden Klebeeigenschaften im Labor unter Beweis stellen konnte, ist es erwähnenswert, dass sein Einbau früh am Morgen auf bereits ausgehärtetem Haftkleber durchgeführt wurde, während andere Systeme (System B und D) auf relativ nicht völlig ausgehärtetem Haftkleber installiert wurden, was den Schichtenverbund beeinträchtigte. Dies gilt insbesondere für System B, bei dem ein sehr hoher Anteil an Haftkleber verwendet wird.

Darüber hinaus wurde, wie der Hersteller von System C hervorhebt, die erforderliche Einbautemperatur der Deckschicht von 140 °C, die für die Sicherstellung der Hafteigenschaften des Systems wichtig ist, nicht erreicht.

Vergleicht man die in-situ-Ergebnisse mit dem Leutner-Laborversuch, so lässt sich folgendes sagen: Obwohl für jedes Bewehrungssystem nur 1 oder 2 Bohrkern aus der Teststrecke getestet wurden, scheint die Rangfolge ähnlich zu sein (System A am besten, gefolgt von System B). Wie in früheren Untersuchungen (Raab et al. 2017) gibt es Systeme, die im Labor ein besseres oder vergleichbares Verhalten wie unverstärkte Proben zeigen, während in situ aufgrund von konstruktiven Einschränkungen und Schwierigkeiten die Verstärkung immer eine Grenzschicht darzustellen scheint, die zu geringeren Verbundeigenschaften zwischen den Schichten führt. Die schlechten Verbundeigenschaften der Systeme C und D wurde auch in den Labortests festgestellt. Hier ist anzumerken, dass es Unterschiede zwischen dem auf der Teststrecke und dem im Labor untersuchten System D gab: Auf der Teststrecke wurde System D abgestreut, während dies im Labor nicht der Fall war. Ein direkter Vergleich ist daher nicht möglich.

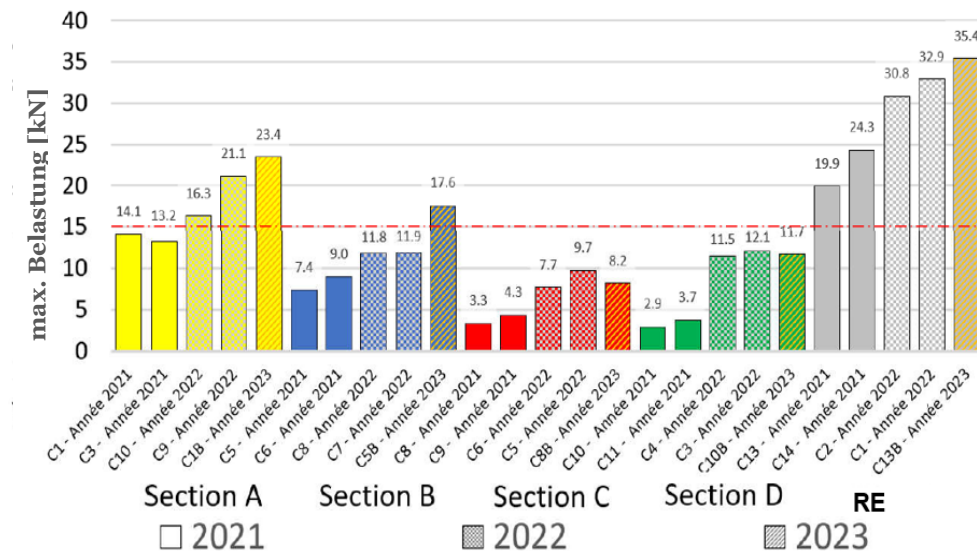


Abbildung 4: Leutner-Ergebnisse der in situ Bohrkerns – Entwicklung 2021/2022 /2023 (Abbildung 73 im Bericht)

Ausbreitung von Rissen MMLS3

Die mit dem MMLS3-Verkehrslastsimulator durchgeführten Rissausbreitungsuntersuchungen zeigten, dass System A besser abschneidet als alle anderen, während System D mit nur etwa 60 % der Lastzyklen, die System A ertragen kann, am schlechtesten ist. Die Messungen der Deformationssensoren (LVDT) zeigten, dass System A dem Belagsaufbau eine hohe Steifigkeit verleiht, während System D am schwächsten ist (siehe Abbildung 5).

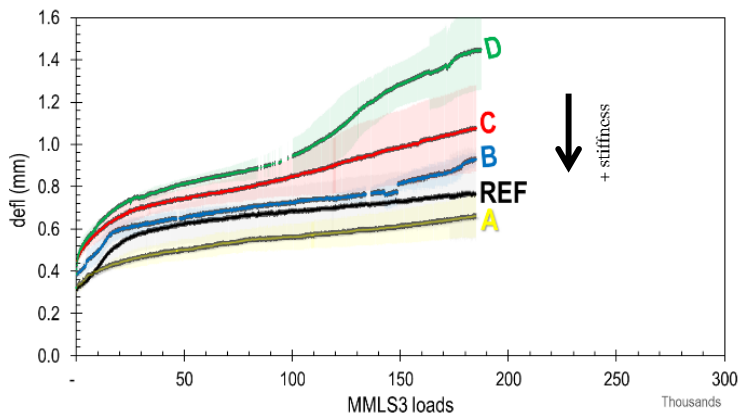


Abbildung 5: Durchschnittliche Deflektionsamplitude vs. Anzahl MMLS3-Zyklen für alle untersuchten Systeme (Abbildung 57 im Bericht)

Es wurden drei verschiedene Mechanismen der Rissbildung und -ausbreitung unterschieden, wie in Abbildung 6 des Berichts dargestellt.

1. Bei einigen Platten zeigt die DIC-Analyse (Digital Image Correlation) unmittelbar nach Beginn der Belastung eine Ablösung der Schichten, gefolgt von der Entstehung eines Risses an der Kerbspitze, der sich durch die Platte bis zur Oberfläche ausbreitet. Dieser Mechanismus wurde bei allen Probekörpern des Systems D und bei 2 Probekörpern des Systems C festgestellt.

2. Beim zweiten Mechanismus beginnt ein vertikaler Riss an der Kerbe, breitet sich direkt an der Grenzfläche aus, was zu einer Trennung der Schichten führt, und setzt sich dann nach oben fort. Dies deutet darauf hin, dass der Schichtenverbund in diesem Fall etwas stärker ist als beim ersten Mechanismus. In beiden Fällen kann sich der vertikale Riss entweder direkt in der oberen Schicht fortsetzen oder sich horizontal verschieben. Eine horizontale Verschiebung deutet auf einen sehr schwachen Schichtenverbund hin, so dass sich die Schichten unter Belastung unabhängig voneinander verhalten. Alle Probekörper des Systems B, ein Probekörper des Systems C und zwei der Referenzprobekörper wiesen diesen Schädigungsmechanismus auf.
3. Der dritte Mechanismus besteht darin, dass sich ein vertikaler Riss direkt in die oberste Schicht ausbreitet, ohne eine Schichttrennung zu verursachen. In den ersten beiden Fällen erfolgte die Rissausbreitung im Allgemeinen schneller als bei Platten, bei denen keine Schichttrennung beobachtet wurde. Dieser Schädigungsmechanismus wurde bei allen Probekörpern des Systems A und einem Referenzprobekörper festgestellt.

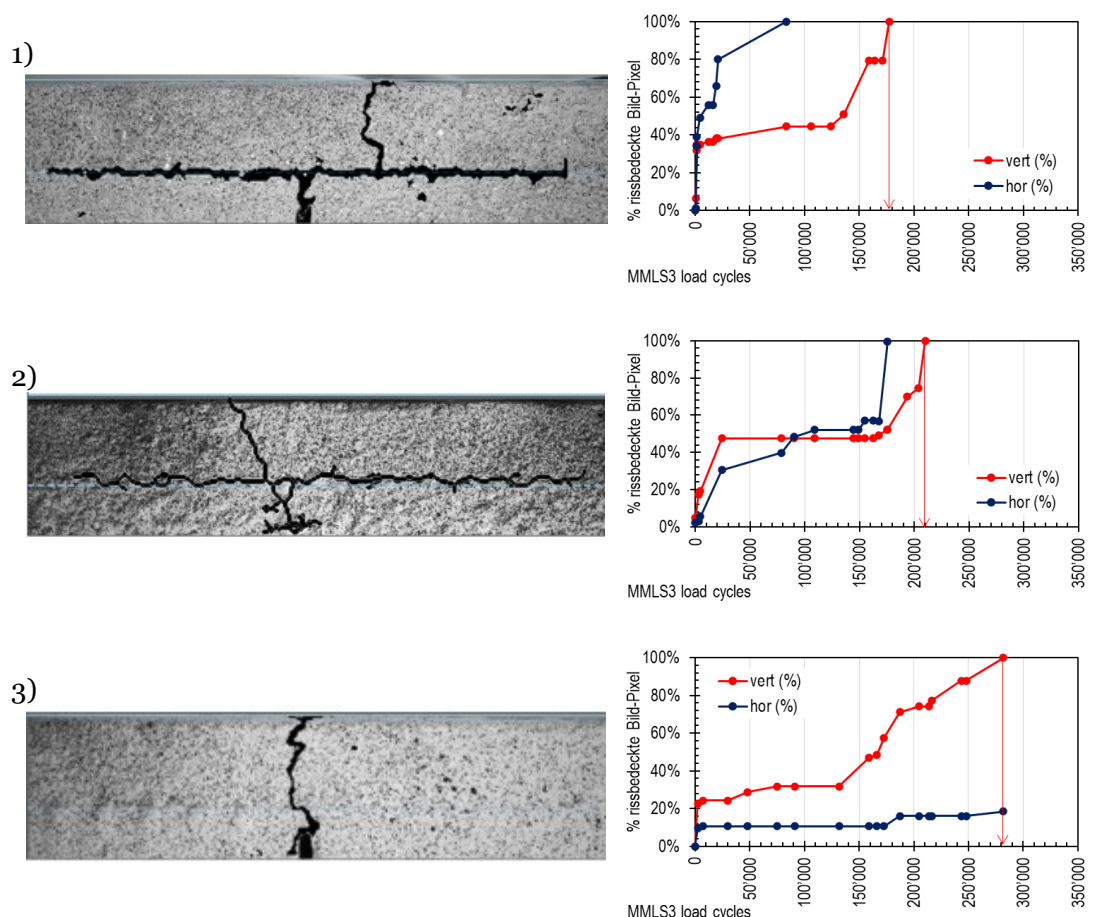


Abbildung 6: Drei Mechanismen der Rissbildung und -ausbreitung, ersichtlich durch die DIC-Analyse (Abbildung 63 im Bericht)

Die Zusammenfassung der Ergebnisse der Rissausbreitung unter MMLS3-Lastzyklen ist in Abbildung 7 dargestellt.

Die Art des Versagens korreliert gut mit der Performance (Anzahl der ertragbaren Lastwechsel) der Platte: Platten, bei denen es zu einer Ablösung zwischen den

Schichten kommt, versagen in der Regel nach weniger Lastzyklen als Platten mit intaktem Schichtverbund. Zusammenfassend haben die Prüfungen gezeigt, dass ein Asphaltbewehrungsgitter die Rissbildung und -ausbreitung wirksam verzögern kann, vorausgesetzt, es gewährleistet einen guten Verbund mit dem Rest der Struktur, was dann zu einem Versteifungseffekt für das gesamte System führt. Folglich zeigen alle Bewehrungssysteme mit Ausnahme von System D ein besseres Verhalten als die unbewehrte Referenz.

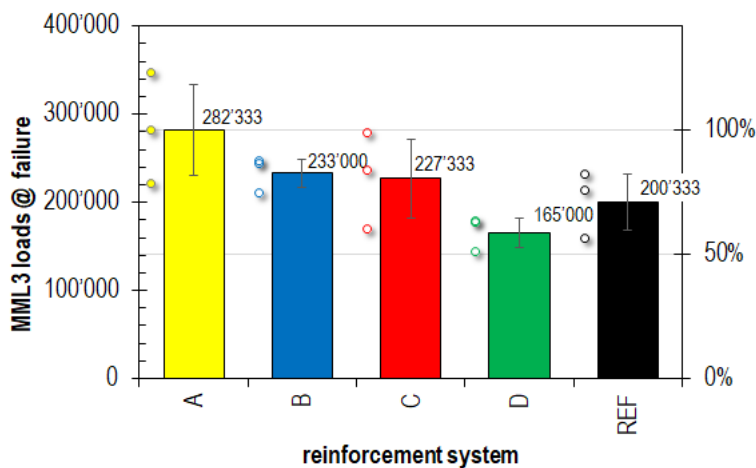


Abbildung 7: Ungefähre Anzahl der Lasten bis zum Versagen der Platten (Abbildung 64 im Bericht)

Rissausbreitung – 4-Punkt-Biegung

Aus den zweischichtigen Platten wurden Balken für die Untersuchung der Rissausbreitung entnommen. Die in Abbildung 8 gezeigten Ergebnisse des 4-Punkt-Biege-Tests (4PB) zeigten, dass alle bewehrten Proben unabhängig vom Gittertyp im Vergleich zu den unverstärkten Referenzproben eine höhere zyklische Belastung ertragen können (farbige Balken).

Die Proben mit Gitterverstärkung B (Cr-B) zeigten die höchste Beständigkeit gegen Rissausbreitung, gefolgt von Proben des Produkts D (Cr-D).

Die Auswirkung von Bewehrungsstößen, wie sie baubedingt auf der Baustelle auftreten (z.B. Bewehrungsgitterstöße entlang der Fahrbahnlänge, d.h. in Längsrichtung, und über die Fahrbahnbreite, d.h. Querrichtung) wurden durch Überlappungsprüfungen untersucht. Diese Prüfung wurde in ähnlicher Weise wie die 4-Punkt Biegung an Proben mit Überlappung der Bewehrungseinlage durchgeführt. Alle Proben wiesen dabei eine Überlappungslänge des Bewehrungsgitters von 15 cm auf.

Die Versuchsergebnisse (vgl. Abb. 8, schraffierte Balken) zeigten, dass die Proben des Systems B (Cr-B) eine höhere Anzahl von Zyklen gegen Rissausbreitung aushielten und sogar eine höhere Beständigkeit im Vergleich zu den Proben mit durchgehendem Bewehrungsgitter (ohne Überlappung) aufwiesen. Dieser Effekt war bei allen verstärkten Proben mit Ausnahme von Produkt D (Cr-B) zu beobachten. Dagegen ist erwähnenswert, dass die Anzahl der Zyklen für Produkt D mit Überlappungslänge noch geringer war als bei der Referenzprobe. Dies könnte mit dem Schichtenverbundproblem von Produkt D zusammenhängen, da die Ergebnisse des

AST-Tests gezeigt haben, dass dieses Produkt sowohl trocken als auch nass eine sehr schlechte Haftfestigkeit aufwies.

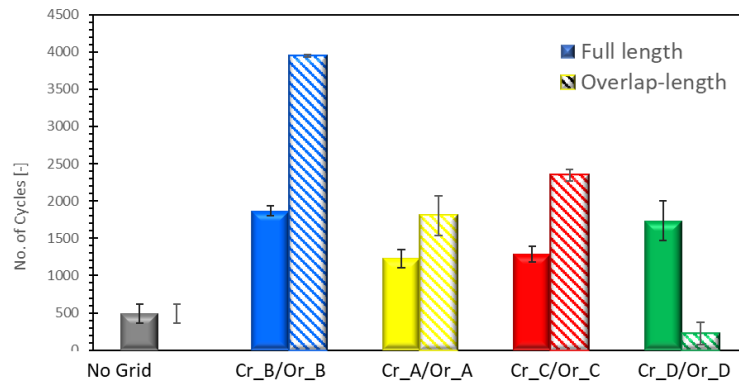


Abbildung 8: Vergleich von verstärkten Gitterproben (volle und überlappende Länge) in Bezug auf die Gesamtzahl der Zyklen (Abbildung 41 im Bericht)

Teststrecke

Am 15.10.2021 wurde der Bau einer Teststrecke auf der Kantonsstrasse T9 Sierre an der Route d'Escala, mit der Stationierung von PR580+740m bis PR590+040m in Sierre (Kanton Wallis) durchgeführt. Parallel zu den Laboruntersuchungen wurden in Abstimmung mit den kantonalen Behörden (Dienststelle für Mobilität, Abteilung Strassen und Verkehr) und der beauftragten Strassenbauunternehmung die Vorbereitungen für die Teststrecke aufgenommen. Die beteiligten Hersteller von Asphaltbewehrungen wurden eingeladen ihre Gitter mit ihren festgelegten Verfahren auf der Teststrecke selbst zu verlegen. Insgesamt wurde die Strecke in fünf verschiedene Abschnitte unterteilt: Abschnitt A, Abschnitt B, Abschnitt C und Abschnitt D (mit Gitterbewehrung) sowie ein Referenzabschnitt ohne Gitterbewehrung. Vor und nach dem Bau der Teststrecke wurden jährlich Messkampagnen zur Zustandserfassung durchgeführt. Jede Messkampagne umfasste die Entnahme von Bohrkernen für die Bestimmung des Schichtenverbundes (Leutner-Prüfung), die Bestimmung der Deflektion mit dem Falling Weight Deflectometer (FWD) sowie die Zustandserfassung mit dem ARAN-Messsystem und in einigen Fällen eine visuelle Inspektion.

Die Zustandserfassung mit dem Messsystem ARAN umfasste die Merkmale Oberflächenschäden (Index I1) sowie Ebenheit in Längs- (Index I2) und Querrichtung (Index I3) nach VSS-Norm 40925 sowie die Makrotexturwerte. Der Index der Oberflächenschäden I0 zeigte für alle bewehrten und unbewehrten Teststreckenabschnitte einen guten bis durchschnittlichen Zustand. Die Ergebnisse des Längsebenheitsindex I2 zeigten für den Referenzabschnitt einen kritischen Zustand sowie eine Mischung aus gutem und durchschnittlichem Zustand. Für Abschnitt B wurde festgestellt, dass sich ein kleiner Teil in einer Richtung in ausreichendem Zustand befand, während der Rest des Abschnitts eine Mischung aus guten und durchschnittlichen Werten aufwies. Der Querebenheitsindex I3 zeigte, dass der Referenzabschnitt und der Abschnitt B überwiegend in gutem Zustand waren, während die Abschnitte A, C und D sich überwiegend in durchschnittlichem Zustand befanden.

Es wurden Messungen mit dem Falling Weight Deflectometer er (FWD) durchgeführt, und die Ergebnisse wurden auf eine Referenztemperatur von 15 °C zurückgerechnet. Die rückgerechneten Elastizitätsmodule der Asphaltbetonschicht, wie in Abbildung 9 dargestellt, zeigten, dass der unbewehrte Abschnitt im Vergleich zu den bewehrten Abschnitten etwas höhere Werte aufwies. Dieser Unterschied könnte auf das Vorhandensein der Bewehrung an der Schichtgrenze zurückgeführt werden, die die Spannungsverteilung beeinflusst haben könnte. Von allen Abschnitten mit Bewehrungsgittern wies die Asphaltschicht mit Produkt A etwas höhere Modulwerte als diejenige der anderen Bewehrungssysteme auf.

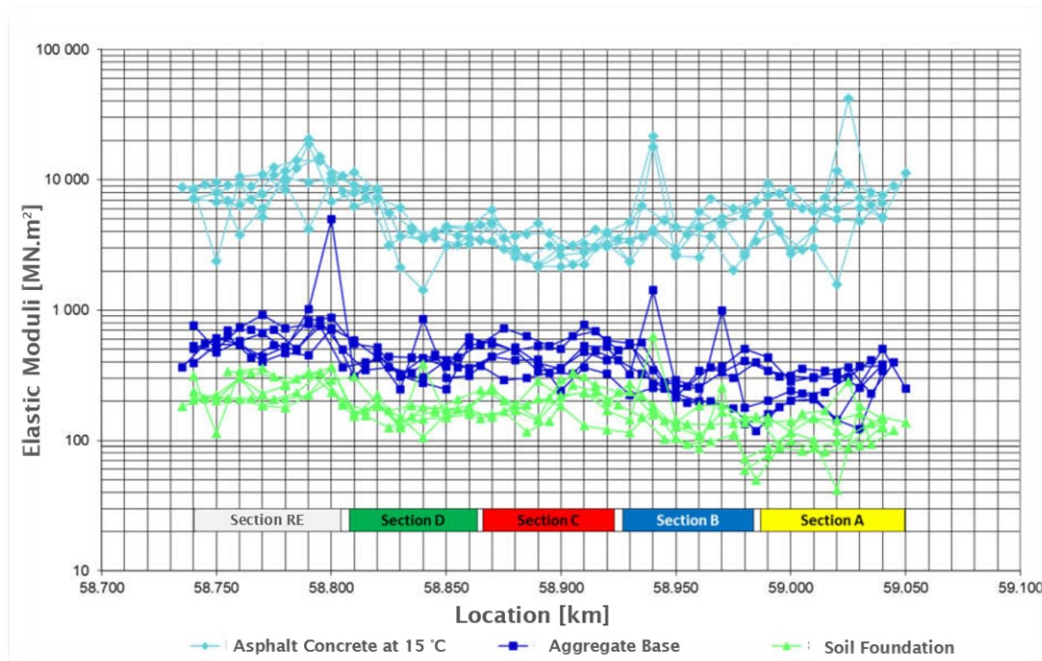


Abbildung 9: Hinten berechnete Elastizitätsmodule der Prüfstrecke (Abbildung 80 im Bericht)

Oberflächenschäden Index I_1

Abschnitt	Abschnitt RE		Abschnitt D		Abschnitt C		Abschnitt B		Abschnitt A	
Richtung	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Mittelwert	0.9	1.1	1.4	1.2	1.4	1.1	0.9	1.3	1.3	1.1
Minimum	0.8	0.8	1.4	0.8	1.4	0.8	0.8	0.8	0.8	0.8
Maximum	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

Tabelle 2: Detaillierte Ergebnisse des Oberflächenschäden Index I_1 (Tabelle A8 im Anhängen)

Ebenheit in Längsrichtung, Index I_2

Abschnitt	Abschnitt RE		Abschnitt D		Abschnitt C		Abschnitt B		Abschnitt A	
Richtung	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Mittelwert	1.1	1.0	1.0	0.5	0.8	0.9	2.2	1.4	1.0	0.7
Minimum	0.3	0.4	0.5	0.0	0.3	0.2	1.5	0.2	0.6	0.4
Maximum	1.6	1.4	1.5	1.0	1.8	2.1	3.2	2.2	1.5	1.0

Tabelle 3: Detaillierte Ergebnisse der Ebenheit in Längs Index I_2 (Tabelle A9 im Anhängen)

Ebenheit in Querrichtung, Index I₃

Section	Abschnitt RE		Abschnitt D		Abschnitt C		Abschnitt B		Abschnitt A	
Abschnitt	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Mittelwert	0.8	1.0	1.4	1.2	1.3	1.2	0.6	1.2	1.3	0.9
Minimum	0.5	0.7	1.0	0.6	1.1	0.8	0.3	0.9	1.0	0.3
Maximum	1.1	1.4	1.6	1.5	1.5	1.7	1.3	1.5	1.5	1.5

Tabelle 4: Detaillierte Ergebnisse der Querrichtung Index I₃ (Tabelle A11 im Anhängen)

Darüber hinaus wurde im Rahmen der Messkampagne im Jahr 2023 die Oberflächenbeschaffenheit hinsichtlich des Rissbildes bewertet. Die Auswertung ergab, dass die Risse im Referenzabschnitt überwiegend in Querrichtung, während die Risse in den bewehrten Profilen überwiegend in Längsrichtung verliefen. Insgesamt war die Anzahl der sichtbaren Risse im Referenzabschnitt höher als in den bewehrten Abschnitten. Für eine genauere Rissbewertung wurden die ARAN-Daten mit Hilfe bildgebender Verfahren weiterverarbeitet. Bei dieser Auswertung wurde die Anzahl der Risse nach 3 Jahren mit der Anzahl der Risse vor dem Bau der Teststrecke verglichen. Zu beachten ist, dass der Vergleich auf der Grundlage der verfügbaren Daten vor dem Bau im Jahr 2019 nur in eine Richtung (d. h. in Richtung Sierre) durchgeführt wurde. Die Balken in Abbildung 10 zeigen die Anzahl der Risse basierend auf den ARAN-Messungen im Jahr 2019 vor und im Jahr 2023, zwei Jahre nach dem Einbau der Teststrecke. Wie aus der Abbildung 10 ersichtlich, ist die Anzahl der Risse im Jahr 2019 (basierend auf der Pixelzahl) im Referenzabschnitt deutlich höher als in allen bewehrten Abschnitten. Die Messungen im Jahr 2023 zeigen auch, dass der Anteil der Risse nach 2 Jahren im Referenzquerschnitt im Vergleich zu den bewehrten Querschnitten immer noch höher ist.

Abbildung 11 zeigt die Anzahl der Risse im Jahr 2023 prozentual zur Anzahl der Risse im Jahr 2019 für alle Abschnitte. Wie bereits erwähnt, findet sich der höchste Prozentsatz mit ca. 61 % im Referenzabschnitt, gefolgt von Abschnitt B mit einem Prozentsatz von 50 %. Abschnitt A weist im Verhältnis zur Anzahl der Risse vor dem Bau die geringste Anzahl von Rissen auf (ca. 9 %), während der Anteil des Abschnitts C bei ca. 25 % und derjenige des Abschnitts D bei 33 % liegt. Der prozentuale Wert von 9 % für Abschnitt A beinhaltet eine Arbeitsfuge, die wieder an der Oberfläche durchgeschlagen ist und daher nicht in Verbindung mit den Bewehrungsgittern gebracht werden darf. Der Prozentsatz von 9 % für den Abschnitt A beinhaltet nicht die Arbeitsfuge, die sich wieder an der Oberfläche gezeigt hat, da diese in keinem Zusammenhang mit Performance steht.

Da für die Bewertung der Wirksamkeit der Bewehrungsgitter die Anzahl der reflektierenden Risse relevant ist, wurde der Prozentsatz der Reflektionsrisse (Vergleich der Anzahl der Risse vor und nach dem Bau) für jeden Abschnitt wie in Abbildung 12 dargestellt bewertet. Auch hier weist der Referenzquerschnitt ohne Bewehrung die höchste Anzahl an Reflektionsrissen (43 %) auf, gefolgt von Abschnitt B mit ca. 20 %. In Abschnitt A liegt die Anzahl der Reflektionsrisse bei ca. 2 % (ohne die Arbeitsfuge, die nicht als reflektierender Riss gilt), während in Sektion C und D keine reflektierenden Risse zu finden sind.

Die Untersuchungen ergaben somit, dass die eingebauten Verstärkungen durchaus in der Lage sind, das Auftreten von reflektierenden Rissen zu verhindern oder zumindest zu verzögern.

Ein Vergleich, der in den verschiedenen Abschnitten eingebauten Bewehrungsgitter untereinander und der Referenz gestaltet sich jedoch schwierig, da eine Inspektion der Teststrecke im Mai 2024 sehr unterschiedliche Geometrien in Form von Kurven und Kreuzungen und damit unterschiedliche Verkehrslasten für die einzelnen Abschnitte ergab. Zu nennen sind in diesem Zusammenhang der Kreisverkehr an der Einmündung zum Abschnitt A und die Abbiegespur im Abschnitt B.

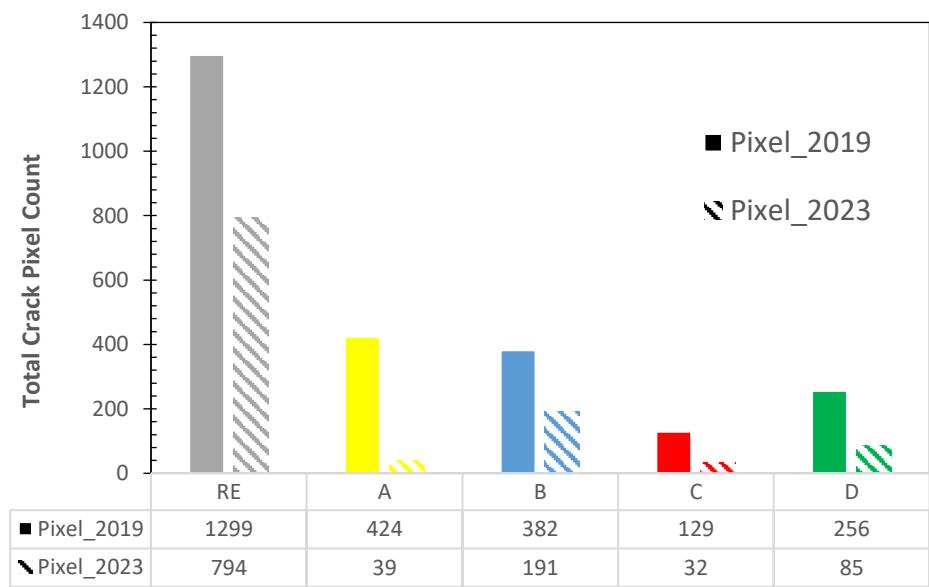


Abbildung 10: Pixelwert der Oberflächenrisse, gemessen mit bildgebender Technik (Abbildung 88 im Bericht)

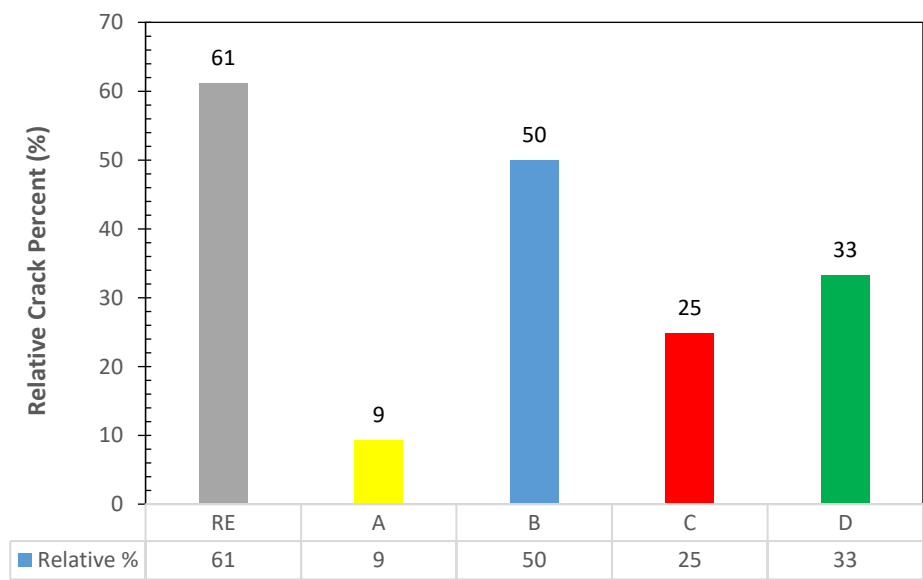


Abbildung 11: Relativer Pixelwert der Oberflächenrisse (Abbildung 89 im Bericht)

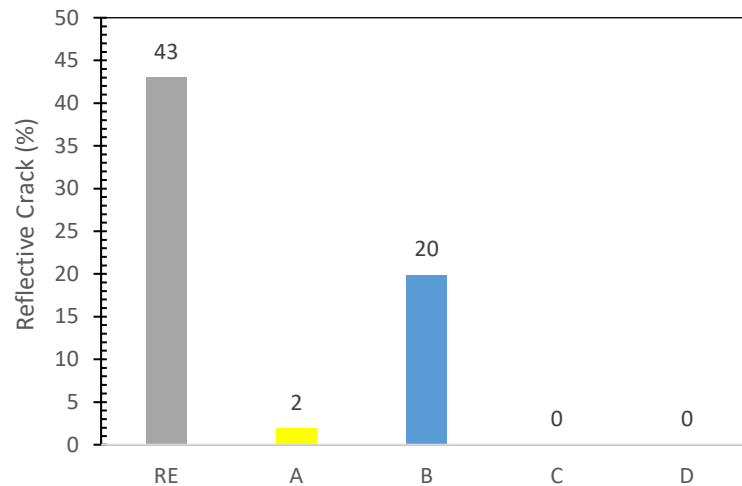


Abbildung 12: Relativer Pixelwert der Reflektionsrisse (Abbildung 90 im Bericht)

FEM-Analyse

Die Modellierung umfassten den Leutner- und den MMLS3-Versuch mit dem Ziel, die Grundlagen für die Berücksichtigung von Asphaltbewehrungen bei der Fahrbahndimensionierung zu schaffen. Zu diesem Zweck wurde die Finite-Elemente-Methode (FEM) als am besten geeignetes Werkzeug gewählt, um die komplexe Geometrie der Gitter und die Wechselwirkung zwischen ihnen und den umgebenden Schichten zu modellieren. Die FEM-Analyse wurde mit Hilfe der kommerziellen Software Abaqus durchgeführt. In einem ersten Schritt wurde ein Modell der Leutner-Prüfung unter Verwendung der Abaqus/CAE-Umgebung erstellt, um das am besten geeignete Modell für den Schichtenverbund unter Berücksichtigung seiner Parameter zu ermitteln. Nach der Prüfung verschiedener Verbundmodelle wurden kohäsive Elemente ausgewählt, um die Wechselwirkung zwischen den Asphaltschichten zu simulieren. Die linke Seite von Abbildung 13 zeigt die Geometrie des Modells und die rechte Seite den Vergleich der Modellergebnisse mit der Messung. Bei geeigneter Wahl der Parameter, kann das Modell das Prüfergebnis recht genau vorhersagen.

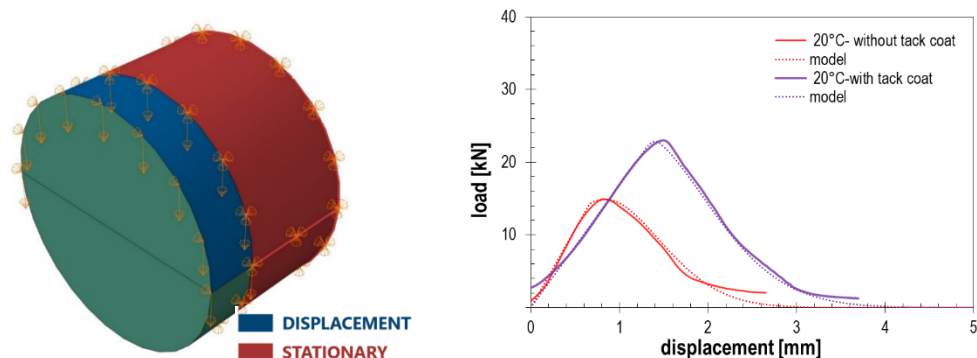


Abbildung 13: FEM-Geometrie des Leutner-Versuchs (links) und Vergleich der mit dem Versuch und dem Modell ermittelten Last-Verschiebungskurven (Bild 107 im Bericht)

Die Simulationen des MMLS3-Tests wurden mit einem in Python programmierten Abaqus-Plug-in erstellt. Das Plug-in zeigt ein Guided User Interface (GUI)-Formular in der CAE-Umgebung von Abaqus (Finite Element Software). In diesem Formular kann der Benutzer alle Parameter der Simulation eingeben, einschliesslich der Geometrie der Platten und der Raster, der Materialien, der Last, der Wechselwirkungseigenschaften zwischen den Schichten und der Verbundparameter. Mit diesem Verfahren ist es möglich, 3D-Simulationen unter beweglichen Lasten, für temperaturabhängige viskoelastische Asphaltmaterialien mit eingebetteten Bewehrungsgittern verschiedener Geometrien und Verbundparameter zu erstellen, einschliesslich der kohäsiven Elemente mit Parametern, die an den vorher erwähnten Leutner-Modellen kalibriert wurden (siehe Abbildung 14). Das Modell wurde unter Verwendung der gemessenen Verschiebungen kalibriert, um eine gute Übereinstimmung mit den Messwerten zu erreichen (siehe Abbildung 15). Wie in Abbildung 16 ersichtlich, zeigen die Ergebnisse der Dehnungen für nicht verstärkte und die gitterverstärkte Situation einen leichten Unterschied. Das entwickelte Modell kann auf ein reales Fahrbahnmodell extrapoliert werden, was auch einen Vergleich verschiedener Bewehrungsgitter ermöglicht, ohne dass zusätzliche Prüfungen erforderlich sind.

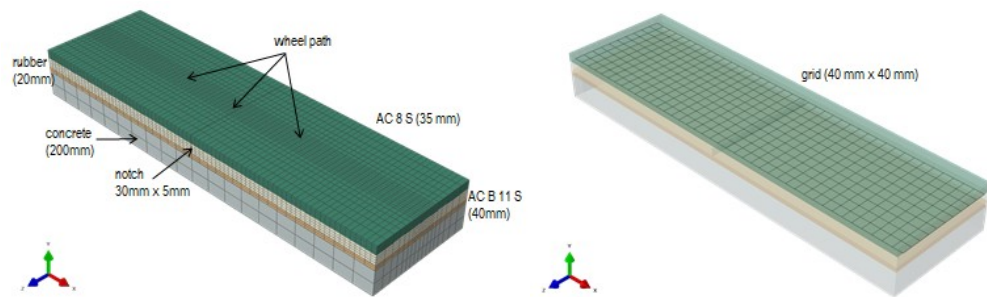


Abbildung 14: Ansicht der Geometrie und des Netzes des FE-Modells (Abbildung 110 im Bericht)

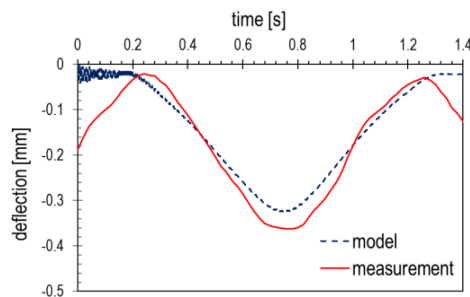


Abbildung 15: Kalibrierung des Modells (Abbildung 114 im Bericht)

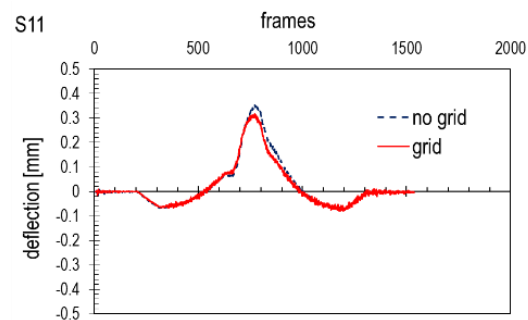


Abbildung 16: Dehnungen in Längsrichtung für ein Element auf der Oberseite der Kerbe, in einem Modell mit und ohne Gitter (Abbildung 116 im Bericht)

Die Validierungsergebnisse bestätigen die Genauigkeit des entwickelten Modells und bieten eine Grundlage für weitergehende Analysen unter Berücksichtigung spezifischer Gittereigenschaften wie Rippenabstand, Materialmodul und Querschnittsfläche.

Zukünftige Arbeiten sollten Leutner-Prüfungen bei verschiedenen Temperaturen zur genaueren Bestimmung der Schichtverbundfestigkeitsparameter und der Werte für kohäsive Elemente umfassen. Mit Hilfe des Modells können anschliessend die Auswirkungen der Lage der Bewehrungsgitter, der Höhe der Last, der Geschwindigkeit und der Temperatur auf die Spannungen und Verformungen im Fahrbelag bestimmt werden. Eine grossmassstäbliche Erweiterung der Simulationen auf Fahrbahnen unter Verkehrslasten wird entscheidende Daten zur Bemessung von verstärkten Fahrbahnen mit mechanistisch-empirischen Ansätzen liefern.

LCCA

Basierend auf den Ergebnissen der vorangegangenen Arbeitspakete wurde eine Lebenszykluskostenanalyse (LCCA) durchgeführt. Die Grundlage für die LCCA bildet ein theoretisches und auf Kennzahlen abgestütztes Nutzungsdauermodell, welches aufgrund der aus dem Projekt vorhandenen Untersuchungsergebnisse für drei verschiedene Szenarien angepasst wurde. Als relevant wurden die MMLS3-Belastungsversuche aufgrund ihres realitätsnahen Testaufbaus erachtet. Dafür wurde als Ausgangssituation von einem bestehenden und an der Fahrbahnoberfläche sichtbar durch Risse geschädigten Asphaltstrassenabschnitt ausgegangen. Die darunter liegenden Asphalttschichten haben bereits den grössten Teil ihrer theoretisch vorgesehenen Lebensdauer verbraucht. Es stellt sich nun die Frage, ob eine grundlegende Erneuerung (E2) in der alle Asphalttschichten erneuert werden oder nur ein Ersatz der Asphaltdeckschicht in Kombination mit einer Asphaltbewehrung als Erhaltungsmassnahme an diesem Abschnitt durchgeführt werden soll. Dafür wurden drei verschiedene Szenarien Ref, V1 und V2 definiert und deren Performance anhand definierter Kennzahlen (Key Performance Indicators: KPIs) verglichen. Das Referenzszenario Ref beinhaltet einen Austausch aller Asphalttschichten zu Beginn des Analysezeitraums. Die Szenarien V1 und V2 bestehen aus dem Austausch der Asphaltdeckschicht mit einer Asphaltbewehrung zwischen Deck- und Binderschicht. Dadurch verschiebt sich die Zeit für den Austausch aller drei Asphalttschichten. Diese zwei Szenarien V1 und V2 bilden, abgeleitet aus den Ergebnissen der MMLS3-Belastungsversuchen, die Bandbreite der vorhandenen Einbauqualität in Kombination mit dem verwendeten Asphaltbewehrungssystem ab.

Szenario V1 geht davon aus, dass während des Einbaus optimale Bedingungen herrschen, die zu einer bestmöglichen Einbauqualität führen. Dadurch konnte der Schichtverbund zwischen Bewehrungsgitter- und Asphalttschicht optimal erreicht werden. Szenario V2 hingegen nimmt an, dass ein optimaler Verbund zwischen dem Bewehrungsgitter und den Asphalttschichten nicht erreicht werden konnte. Dies führt in der Regel dazu, dass die auf die Asphaltdeckschichten einwirkenden Verkehrslasten nicht optimal auf die darunterliegenden Schichten übertragen werden können und die Asphaltdeckschicht vorzeitig ermüdet.

Der Nutzen jeder Variante wurde für jeden KPI berechnet und anhand der Summe der Flächen unter den KPI-Kurven quantifiziert. Die entsprechenden Kosten beinhalten die rein massnahmenbedingten Kosten. Die drei Modellvarianten zeigen ähnliche Vorteile in Bezug auf die Performance der vier KPIs. Das Referenzszenario (Ref) liefert eine Gesamtfläche von 869,5 Einheiten, Szenario V1 eine Gesamtfläche von 867,9 Einheiten und Szenario V2 eine Gesamtfläche von 866,6 Einheiten. Bei den Gesamtkosten gibt es jedoch einen deutlich grösseren Unterschied zwischen den drei Modellvarianten. Szenario V2 ist mit Gesamtkosten von 208.687 CHF die schlechteste Variante, da bei diesem Szenario innerhalb des Analysezeitraums eine zusätzliche Erhaltungsmassnahme (E2) durchgeführt werden muss. Das Referenzszenario Ref mit 165.672 CHF und Szenario V1 mit 147.965 CHF lieferten niedrigere Kosten.

Die Ergebnisse dieses Anwendungsbeispiels zeigen, dass der Einsatz von Asphaltbewehrung über einen Betrachtungszeitraum von 50 Jahren einen zusätzlichen Nutzen bieten kann. Wird die Asphaltbewehrung nicht optimal eingebaut oder werden die Anforderungen an den Schichtenverbund nicht erfüllt, kann eine zusätzliche Erhaltungsmassnahme notwendig werden, was zu erhöhten Kosten innerhalb des

Lebenszyklus führt. Allerdings kann der Einsatz von Asphaltbewehrung im Anwendungsfall im Vorteil sein, wenn der technisch sinnvolle Austausch aller Asphaltsschichten aus verkehrstechnischen, wirtschaftlichen, personellen oder anderen Gründen vorübergehend nicht durchführbar ist. Zusätzlich wurde untersucht, welchen Einfluss eine notwendige Deponierung von Asphaltfräsgut mit Bewehrungsgitterkontamination hat. Je nach den produktspezifischen Umweltproduktdeklarationen (engl. Environmental Product Declarations EPDs), die zur Bewertung der CO₂-Äquivalenzeinheiten und anderer Nachhaltigkeitsindikatoren verwendet werden, kann dies zu einer weiteren Verringerung des Nutzens führen.

Diskussion

Basierend auf den dargestellten Ergebnissen dieses Projekts ergeben sich folgende Diskussionspunkte:

- Die Untersuchung des Schichtenverbunds zeigt, dass es Asphaltbewehrungssysteme gibt, die im Labor zu einem besseren oder vergleichbaren Verhalten wie unverstärkte Proben führen, während in situ aufgrund von konstruktiven Zwängen und Schwierigkeiten die Asphaltbewehrungsgitter immer eine Diskontinuität darstellen, die oft zu geringeren Verbundeigenschaften zwischen den Schichten führt.
- Der Schichtverbund in-situ verbessert sich im Laufe der Zeit deutlich, eine Tatsache, die mit früheren Untersuchungen übereinstimmt (Raab et al., 2017, Raab & Partl, 2007). Einige Hersteller (Asphaltbewehrungsgitter mit selbstklebender Membran) behaupten sogar, dass dieser Mechanismus (Verdichtung durch Verkehrslast in Kombination mit hohen Temperaturen im Sommer) die Performance ihres Produkts verbessert.
- Der Vergleich zwischen MMLS3- und 4PB-Rissausbreitungsprüfungen erwies sich als nicht einfach. Dies könnte auf die unterschiedliche Herstellung der Proben und die unterschiedlichen Probekörpermassstäbe zurückzuführen sein. Darüber hinaus war die Testtemperatur ein entscheidender Faktor, da die MMLS3-Proben bei 20 °C geprüft wurden, während der 4PB-Prüfung bei 5 °C durchgeführt wurde.
- Insgesamt muss die MMLS3-Prüfung aufgrund ihrer Belastungskonfiguration und der Grösse und Geometrie der Proben, insbesondere im Hinblick auf die Maschenweite der Verstärkungsprodukte, als realistischer und mit der in-situ-Situation vergleichbar angesehen werden. Darüber hinaus ist es erwähnenswert, dass der 4PB-Rissausbreitungsprüfung pro Aufbauvariante nur an zwei Proben durchgeführt wurde.
- Die Proben mit einer Überlappungslänge von 15 cm erwiesen sich im 4PB-Test als widerstandsfähiger als die in voller Länge geprüften Proben. Dies zeigt, dass bei einer den Herstellerempfehlungen entsprechenden Überlappung keine Probleme zu erwarten sind und die Überlappungslänge für ein bestimmtes Bewehrungsprodukt sogar optimiert werden könnte.
- Die gewählte Teststrecke war Teil des planmässigen Unterhaltsprogramms im Kantons Wallis und wurde in Zusammenarbeit mit den kantonalen Behörden (Dienststelle für Mobilität, Abteilung Strassen und Verkehr) ausgeführt. Obwohl sie aufgrund des hohen Schwerverkehrsaufkommens ausgewählt worden war, waren die Geometrie (Kurven), das Verkehrsaufkommen und das Verkehrsregime (Anzahl

schwerer Lkw, Abbiegespuren, Querkräfte durch Bremsen im Kreisverkehr) und die Schäden (Anzahl der Risse) vor der Sanierung für alle Abschnitte sehr unterschiedlich. Wie bei einer Zustandserfassung im Jahr 2024 zu sehen war, wurden Abschnitte mit Kornausbrüchen und Rissbildung in der Deckschicht gefunden, Phänomene, welche nicht notwendigerweise in Zusammenhang mit der Wirksamkeit von Asphaltbewehrungen gebracht werden können, worin sich aber die unterschiedlichen Beanspruchungen der einzelnen Streckenabschnitte zeigen.

- In Bezug auf die Installation der verschiedenen Asphaltbewehrungsgitter ist erwähnenswert, dass der Einbau von System A am frühen Morgen auf bereits ausgehärtetem Haftkleber erfolgte, während andere Systeme (System B und D) auf einem nicht völlig ausgehärteten Haftkleber installiert wurden. Dies gilt insbesondere für System B, bei dem ein sehr hoher Anteil an Haftkleber verwendet wird. Darüber hinaus wurde, wie der Hersteller eines Systems mit selbstklebender Membran hervorhebt, die erforderliche Temperatur der Deckschicht von 140 °C, die dieser für wichtig hält, um die Verbundeigenschaften seines Systems zu gewährleisten, nicht erreicht.
- Ein Vergleich zwischen Labor- und in-situ-Ergebnissen war für das Bewehrungsprodukt D nicht möglich, da der Lieferant beim Einbau des Testfeldes eine Abstreuerung des Bewehrungsgitters verwendete, diese jedoch bei den Laborproben nicht zum Einsatz kam.

Schlussfolgerungen und Empfehlungen

- Wie oben erwähnt, hat die Prüfung des Schichtenverbunds ergeben, dass es Asphaltbewehrungssysteme gibt, die im Labor zu einem besseren oder vergleichbaren Verhalten wie der Referenzaufbau ohne Asphaltbewehrung führen. In-situ zeigte sich aufgrund von konstruktiven Einschränkungen und Schwierigkeiten, dass die Asphaltbewehrungsgitter zwischen Deck- und Binderschicht immer eine Diskontinuität darstellen, was dann auch oft zu geringeren Verbundeigenschaften führt. Dennoch gibt es Asphaltbewehrungssysteme, die den Anforderungswert von 15 kN nach Schweizer Norm erreichen können. Aus diesem Grund ist es sehr wichtig, dass die Hersteller einerseits ihre Systeme hinsichtlich des Schichtenverbunds weiter verbessern und andererseits den Einbau ihrer Produkte so weit wie möglich optimieren.
- Auftraggeber wie Gemeinden, Kommunen, Länder, Bund etc. und Bauunternehmer müssen ihrerseits sicherstellen, dass die notwendigen Einbaubedingungen bei der Anwendung von Asphaltbewehrungssystemen konsequent eingehalten werden und dass alle Voraussetzungen für einen optimalen Einbau eines spezifischen Asphaltbewehrungsgitters erfüllt sind.
- Durch den Einsatz unterschiedlicher Laborprüfverfahren zur Bestimmung der Rissausbreitung (MMLS3 und 4PB) konnte die rissüberbrückende und risshemmende Wirkung von Asphaltbewehrungsgittern und der Unterschied zu Systemen ohne den Einsatz von Asphaltbewehrung für verschiedene Asphaltbewehrungssysteme nachgewiesen werden. Auch in dieser Hinsicht wurde festgestellt, dass der Schichtenverbund einer der bestimmenden Faktoren für die rissüberbrückende

Wirkung eines bewehrten Fahrbahnsystems ist. Um eine bessere Performance zu erzielen, ist es daher sehr wichtig, den Verbund zwischen den Schichten zu verbessern, was auch für die Vermeidung von Feuchtigkeitsschäden von entscheidender Bedeutung ist.

- Es wurde gezeigt, dass die Kombination aus labormassstäblichen Prüfungen mit einem Verkehrslastsimulator (MMLS-Tests), Durchbiegungs-messungen mit Verformungsmessern (LVDTs) und digitaler Bildkorrelationsdatenanalyse (DIC) ein wichtiges Werkzeug ist, um den Mechanismus von Asphaltbewehrungssystemen zu verstehen, insbesondere mit der neu vorgeschlagenen Methode zur Bestimmung von horizontalen und vertikalen Rissausbreitungsmustern. Die Auswertung der horizontalen und vertikalen Rissausbreitung zeigt, dass die Wirksamkeit der Asphaltbewehrung von unterschiedlichen Mechanismen abhängt und dass es irreführend sein kann, sich nur auf kleinmassstäbliche Versuche zu verlassen.
- Die Ergebnisse der Überlappungslängentests zeigten, dass die Überlappung der Asphaltbewehrungsgitter, wenn sie gemäss den Herstellerempfehlungen durchgeführt wird, kein Problem darstellt und dass die Überlappungslänge für ein bestimmtes Bewehrungsprodukt sogar optimiert werden kann.
- Hinsichtlich der durchgeführten Prüfungen muss die Rissausbreitungsprüfung mit dem Verkehrslastsimulator MMLS3 im Vergleich zur 4PB-Prüfung aufgrund ihrer Lastkonfiguration und der Prüfkörpergeometrie, insbesondere auch hinsichtlich der Maschenweite der verwendeten Bewehrungsgitterprodukte für die Asphaltbewehrung, als realistischer und mit der in-situ-Situation vergleichbarer angesehen werden. Obwohl die 4PB-Rissausbreitungsprüfung ihre Grenzen hat, ist sie einfacher durchzuführen und somit nützlich für den Vergleich verschiedener Produkte. Um das Konfidenzniveau der Prüfergebnisse zu erhöhen, ist es wichtig, die kleinmassstäblichen Prüfungen mit einer grösseren Anzahl von Prüfkörpern (nicht nur 2 Proben) durchzuführen.
- Bei der Betrachtung der Teststrecke zeigten visuelle Auswertungen des Oberflächenzustands, dass die Anzahl der Risse im Referenzabschnitt im Vergleich zu den bewehrten Abschnitten höher war. Die Risse im Referenzquerschnitt verliefen meist in Querrichtung, während die Risse in den bewehrten Profilen überwiegend in Längsrichtung verliefen.
- Für eine genauere Rissbewertung wurden die mit dem ARAN-Messsystem erfassten Oberflächendaten mit Hilfe bildgebender Verfahren aufbereitet. Nach dieser Auswertung wurde die Anzahl der Risse verglichen und anhand der Anzahl der Risse vor dem Einbau der Teststrecke bewertet. Hier zeigte sich, dass die Anzahl der Risse bereits vor dem Einbau im Referenzabschnitt im Vergleich mit allen bewehrten Abschnitten deutlich höher war. Bei der Berechnung des prozentualen Anteils der Risse vor und 3 Jahre nach Einbau war die Anzahl der Risse im Referenzabschnitt jedoch immer noch höher als die Anzahl der Risse in den bewehrten Abschnitten.
- Für den Referenzabschnitt wurde mit ca. 61 % der höchste Prozentsatz festgestellt, gefolgt von Abschnitt B mit einem Anteil von 50 %. Abschnitt A wies die geringste

Anzahl von Rissen im Verhältnis zu den Rissen vor Einbau auf (ca. 9 %), während der Prozentsatz für die Abschnitte C und D 25 % bzw. 33 % betrug. Darüber hinaus enthielt der Prozentsatz von 9 % für Abschnitt A eine Arbeitsfuge, die wieder an der Oberfläche durchgeschlagen hat und daher nicht in Verbindung mit der Wirkung von Asphaltbewehrungsgittern gebracht werden darf.

- Da für die Bewertung der Wirkung von Asphaltbewehrungsgitter die Anzahl der Reflektionsrisse relevant ist, wurde der Prozentsatz dieser Risse (Vergleich der Anzahl der Risse vor und nach dem Einbau) für jeden Abschnitt bewertet. Auch hier wies der Referenzabschnitt die höchste Anzahl Reflektionsrisse (43 %) auf, gefolgt von Abschnitt B mit 20 %. In Abschnitt A betrug die Anzahl der Reflektionsrisse 2 %, während in den Abschnitten C und D keine Reflektionsrisse festgestellt werden konnten.
- Die in-situ-Untersuchungen zeigten daher, dass die eingebauten Asphaltbewehrungen durchaus in der Lage sind, das Auftreten von Reflektionsrissen zu verhindern oder zumindest zu verzögern.
- Der Vergleich der verschiedenen Abschnitte mit Asphaltbewehrung untereinander und mit der Referenz ohne Asphaltbewehrung war jedoch schwierig, da wie im Mai 2024 anlässlich einer Begehung festgestellt wurde, sehr unterschiedliche Geometrien in Form von Kurven und Kreuzungen und damit unterschiedliche Verkehrslasten für die einzelnen Abschnitte vorlagen. Zu erwähnen sind in diesem Zusammenhang der Kreisverkehr an der Einmündung zum Abschnitt A und die Abbiegespur im Abschnitt B.
- Alle im Rahmen des Projekts durchgeführten Auswertungen und Prüfungen haben ergeben, dass die Anzahl der Labor- und in-situ-Probekörper erhöht werden muss, um verlässliche Ergebnisse präsentieren und Empfehlungen für eine Standardisierung und Anwendung geben zu können.
- Das im Rahmen dieses Forschungsvorhabens entwickelte Vorgehen zur FEM-Analyse ermöglicht es, die Performance verschiedener Asphaltbewehrungssysteme anhand von Produktdaten zu bestimmen, die von den jeweiligen Herstellern zur Verfügung gestellt werden müssen. Mit diesem Vorgehen könnte der Prüfaufwand erheblich reduziert und unterschiedliche Prüfbedingungen einfach bewertet werden.

Résumé (Français)

Introduction

Les chaussées en enrobés bitumineux exposées à un trafic élevé et aux conditions météorologiques atteignent leurs limites de capacité portante au fil du temps et présentent divers types de dommages (fissures, ornières, etc.) qui affectent la sécurité et le confort des usagers de la route. Par conséquent, il est important de rechercher des méthodes d'entretien et de réparation efficaces qui permettent de réduire les coûts de maintenance et d'augmenter la durée de vie, ainsi que d'économiser des matériaux et de l'énergie. Il existe divers types de mesures pour l'entretien de l'infrastructure routière ; les techniques de renforcement devenant de plus en plus populaires, notamment pour prévenir la fissuration prématurée. Bien que la littérature internationale montre qu'en général, les grilles de renforcement ont un effet positif sur les performances de la chaussée, de nombreuses questions restent encore ouvertes, notamment en ce qui concerne leur impact sur la liaison intercouche. Par ailleurs, le projet de recherche ASTRA 2011/011 a fourni des résultats préliminaires très intéressants sur l'utilisation des grilles et les améliorations de performances associées (résistance et durabilité). De leur côté, les fabricants des grilles ont mené des recherches sur leurs matériaux et ont acquis un certain savoir-faire qui leur permet de développer leurs solutions et d'intégrer avec succès leurs produits sur les chantiers. Cependant, ces études utilisent indépendamment des méthodes et des tests différents, et les résultats obtenus ne sont en principe valables que pour le produit spécifique en question.

Bien que l'effet des grilles de renforcement ait été prouvé dans la pratique dans de nombreux cas, il n'existe actuellement aucune théorie fiable pour décrire les mécanismes et l'efficacité de leur utilisation. Par conséquent, il est nécessaire de disposer de méthodes d'investigation et d'essai exhaustives qui permettent une comparaison directe des différentes grilles de renforcement ainsi que de leur efficacité globale et de leur impact sur les performances de la chaussée.

Afin de combler cette lacune, le projet de recherche actuel a évalué les performances de différentes grilles de renforcement à l'aide de divers tests en laboratoire basés sur différentes tailles et dimensions d'éprouvettes, ce qui a permis de mieux comprendre l'utilisation des grilles et en déduire des éléments de bonne pratique. En parallèle, des planches d'essais in situ ont été construites pour suivre et relier les performances sur le terrain des grilles de renforcement par rapport aux évaluations en laboratoire.

Objectifs

Le projet visait à étudier et à promouvoir les bonnes pratiques pour l'utilisation des grilles de renforcement dans les chaussées bitumineuses en fournissant la base méthodologique et normative nécessaire afin de les prendre en compte dans un processus de conception et évaluer leur efficacité (performance). La base générale du projet n'était pas liée à un produit spécifique et c'est pourquoi les principaux fournisseurs de grilles de renforcement opérant en Suisse ont été pris en compte ; les résultats de ce travail étant présentés ici de manière anonyme.

Méthodologie

Les méthodes appliquées dans ce projet se sont appuyées sur des investigations en laboratoire et sur le terrain ainsi que sur une analyse du cycle de vie prenant en compte notamment les différents résultats d'essais. Les principales investigations en laboratoire ont été effectuées à la BFH et à l'Empa. Afin de produire les échantillons nécessaires aux études en laboratoire à la BFH, les informations de production ont été documentées, puis une démonstration pratique de la méthode d'installation des grilles de renforcement des différents fournisseurs a été effectuée dans le laboratoire de la BFH. Par la suite, les échantillons d'essai ont été produits selon les instructions reçues. Étant donné que les échantillons testés à l'Empa étaient d'une plus grande ampleur, les fournisseurs de grilles de renforcement ont été invités à installer leurs produits. La préparation de tous les échantillons de laboratoire a été réalisée conformément aux exigences et directives des fournisseurs d'asphalte, en utilisant les matériaux qu'ils ont fournis.

Les différentes étapes de l'étude du projet sont présentées à la figure 1. Après une mise à jour des connaissances et l'étude de la littérature nationale et internationale récente (Tâche 1), une phase expérimentale a été menée en laboratoire (Tâche 2). Ces investigations comprenaient, entre autres, l'évaluation des composites grille/enrobé (adhérence entre les couches) et l'étude en termes de résistance à la fatigue et à la fissuration (essais de flexion 4 points et de simulateur de trafic) ainsi que l'évaluation de l'impact de la longueur de recouvrement. Les planches d'essais en vraie grandeur (Tâche 3) se sont concentrées sur les aspects pratiques de l'installation des grilles de renforcement et le suivi de leurs performances in situ selon les sollicitations dues aux contraintes mécaniques et climatiques dans des conditions réelles de trafic. À cette fin, des essais sur le terrain avec des campagnes de mesure comprenaient des mesures de relevé automatisé des chaussées avec véhicule spécifique (ARAN), mesures avec déflectomètre à masse tombante (FWD) ainsi que le prélèvement de carottes en vue d'analyses en laboratoire. Les différentes données recueillies en laboratoire et in situ ont ensuite permis d'affiner et d'actualiser la modélisation des chaussées avec renforcement intercouches (Tâche 4). Pour finir, une analyse du coût du cycle de vie (ACV) a été réalisée afin d'intégrer les résultats de toutes les étapes du projet (Tâche 5). Grâce à cette analyse, une méthodologie exhaustive et des recommandations détaillées ont été élaborées pour la conception et la réalisation d'activités de maintenance et de réhabilitation qui impliquent des grilles de renforcement. Dans la tâche 5, les données accumulées et les leçons apprises ont finalement été utilisées pour élaborer une méthodologie détaillée et des recommandations pour la conception et la mise en œuvre d'activités d'entretien/réhabilitation intégrant des grilles de renforcement tout en tenant compte les conditions in situ (conditions climatiques, état de la couche en place, conditions d'installation, etc.).

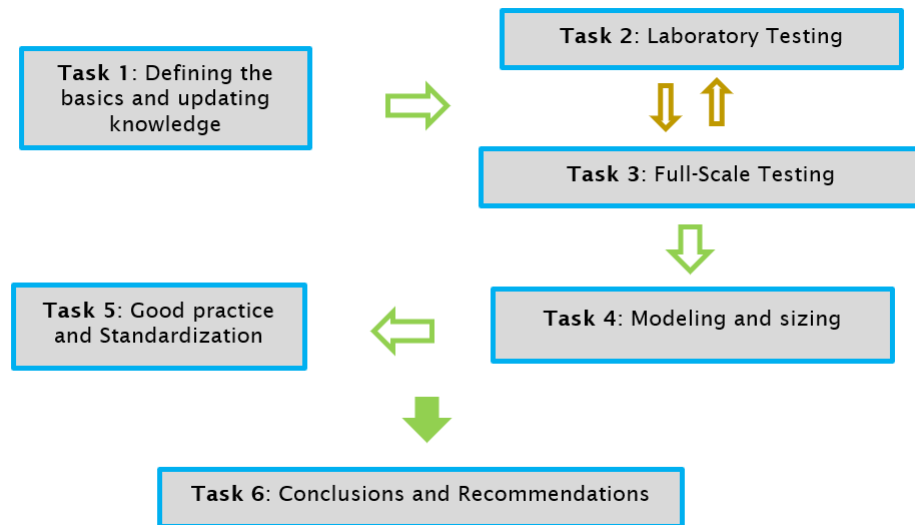


Figure 1 : Différentes étapes du projet (Figure 6 dans le rapport)

Matériaux

Pour l'ensemble des essais effectués dans le cadre de ce projet, les mêmes enrobés bitumineux AC 8 S (couche de surface) et AC B 11 S (couche de fondation) ont été utilisés. Les mélanges ont été obtenus à partir d'une piste d'enrobage à Hinwil. La couche d'accrochage utilisée pour l'encollage était quant à elle une émulsion cationique modifiée par un polymère avec une teneur en bitume résiduelle de 60 %. La grille de renforcement C comportait une membrane auto-adhésive et ne nécessitait donc pas de couche d'accrochage.

Quatre types différents de grilles de renforcement ont été utilisées. Le tableau 1 récapitule les propriétés mécaniques les plus pertinentes de ces grilles. Pour garantir l'anonymat des résultats, une nomenclature A, B, C et D a été utilisée pour les différentes grilles.

Grilles de renforcement

Produit	A		B		C		D	
Couche d'accrochage – Terrain (résiduel) [g/mm ²]	180		690		Aucun*		420	
Couche d'accrochage – Labo (résiduel) [g/m ²]	180		480		Aucun		250	
Direction des fibres	L	T	L	T	L	T	L	T
Matériau des fibres	Verre	Carbone	Animal domestique		Verre		Verre	Carbone
Maille [mm]	15	15	40		10		18	18
Résistance à la traction [N/m]	120	200	50		100		120	200
Allongement [%]	3	1.5	12		3		3	1.5

*L'émulsion a été appliquée uniquement sur les bords de la grille (600 g/m²)

Tableau 1 : propriétés mécaniques de la grille de renforcement utilisée dans le projet (tableau 2 dans le rapport)

Résultats

Liaison intercouches - laboratoire

L'étude en laboratoire a consisté à évaluer des échantillons composites préparés à différentes échelles. Des éprouvettes composites avec double couche d'enrobé ont été préparées à l'Empa et à la BFH pour l'évaluation de l'adhérence intercouche et de la propagation des fissures. Des carottes ont été prélevées sur les échantillons construits pour l'étude de l'adhésion intercouche à l'aide de l'essai standard Leutner et de l'essai de cisaillement avancé (AST).

L'évaluation de la liaison intercouche à l'aide de l'essai Leutner effectué sur des éprouvettes de laboratoire a montré que presque tous les systèmes de grilles de renforcement, à l'exception du A, ont une influence négative sur la liaison intercouche. Dans l'étude en laboratoire, les forces de cisaillement maximales des systèmes renforcés étudiés différaient considérablement. En particulier, le système A a obtenu des valeurs très élevées avec une moyenne de 33,2 kN, suivi par le système B avec une valeur moyenne de près de 25,8 kN, tandis que le système C a obtenu des résultats sensiblement moins bons avec une force de cisaillement maximale moyenne inférieure à 6.0 kN. Le système D a atteint une moyenne de 16,0 kN, mais compte tenu de l'écart-type, la valeur requise de 15 kN selon la norme suisse (SN-EN-12697-48) n'a pas pu être

atteinte pour l'ensemble des éprouvettes testées. Le système A a même obtenu des valeurs plus élevées que le système sans renforcement (référence) qui a atteint une force de cisaillement maximale moyenne de 30,3 kN. Les résultats sont visibles dans la figure 2.

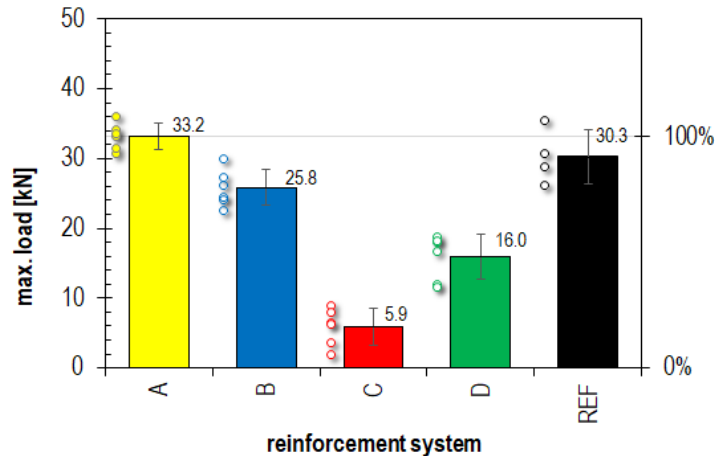


Figure 2 : Résultats du laboratoire d'essai Leutner pour chaque système de renforcement (figure 15 dans le rapport)

Les résultats des essais en laboratoire à l'aide de l'essai AST ont montré que l'adhésion intercouche pour les échantillons de référence (sans grilles) présente une résistance plus élevée à la rupture comparativement aux échantillons avec grilles de renforcement. Comme le montre la Figure 3, les éprouvettes avec grille de renforcement A ont la résistance la plus élevée à la rupture par rapport aux autres éprouvettes renforcées. De plus, après un stockage sous l'eau, le produit A présente une résistance plus élevée à la rupture à l'interface que les autres produits. La sensibilité à l'eau a également été évaluée à l'aide de la procédure d'essai AST. L'effet de l'humidité a réduit la force de rupture pour la référence et les échantillons avec grille A. L'effet de l'humidité s'est avéré non significatif pour les échantillons préparés avec les grilles B, C et D.

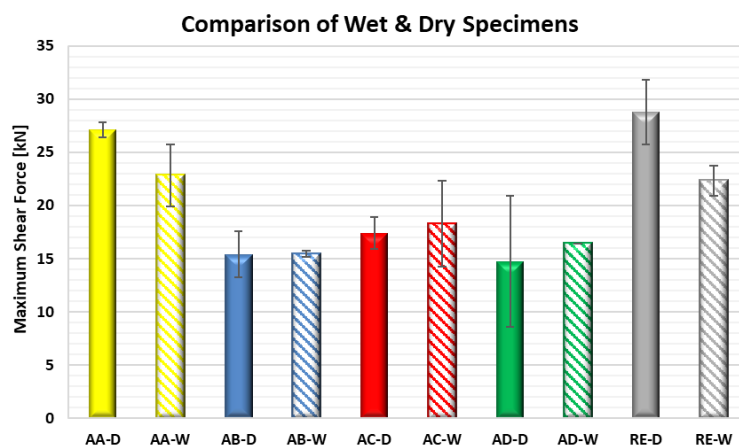


Figure 3 : Force de cisaillement maximale AST, comparaison des échantillons testés à sec et sur sol mouillé (Figure 23 dans le rapport)

Liaison intercouches – in situ

L'étude des carottes prélevées in situ a été réalisée à l'aide de l'essai de liaison intercouche (essai Leutner). Les résultats des essais présentés à la figure 4 ont montré que les carottes issues de la section de référence non renforcée présentaient les valeurs de liaison intercouche les plus élevées par rapport aux sections renforcées, ceci quel que soit le type de grille de renforcement utilisée et donc les meilleures propriétés de collage intercouche. D'autre part, le produit utilisé dans la section A présentait les valeurs les plus élevées, suivi de la section B, tandis que les sections C et D ne semblent jamais être en mesure d'atteindre l'exigence normative de 15 kN. D'après les résultats obtenus, il est clair que l'état d'adhésion intercouche s'est amélioré au fil du temps, un fait qui est en adéquation avec les études antérieures (Raab et al., 2017, Raab & Partl, 2007). L'augmentation de la force de cisaillement pour les systèmes A, B et de référence est de l'ordre de 40 %, tandis que l'augmentation pour les systèmes C et D est respectivement de 66 % et 76 %. Bien que le système A ait démontré des qualités de liaison supérieures en laboratoire, il convient de noter que son installation a été réalisée tôt le matin sur une couche de liaison déjà durcie, tandis que d'autres systèmes (par exemple, les systèmes B et D) ont été installés sur une couche de liaison relativement non durcie, ce qui a donc affecté la qualité de collage entre couches. Ceci est notamment le cas pour le système B qui utilise un taux d'adhérence très élevé. De plus, comme l'a souligné le fournisseur du système C, la température requise de 160 °C pour la couche de surface, qui est importante pour garantir les propriétés d'adhérence du système, n'a pas été systématiquement atteinte.

En comparant les résultats sur le terrain avec l'essai Leutner en laboratoire, on peut observer ce qui suit : Bien que seulement 1 ou 2 carottes prélevées in situ aient été testées pour chaque système de renforcement, le classement semble être similaire (le système A est le plus performant, suivi du système B). Comme dans les études précédentes (Raab et al. 2017), il existe des systèmes qui montrent en laboratoire un comportement meilleur ou comparable à celui des échantillons non renforcés, tandis qu'in situ, en raison des contraintes et des difficultés de construction, le renforcement semble toujours fournir une barrière entre les couches, ce qui conduit à des propriétés de liaison intercouches péjorées. Les performances moins bonnes des systèmes C et D ont également été constatées lors des tests en laboratoire, tandis qu'il convient de noter que le système D testé in situ (avec écaillage) était différent du système D testé en laboratoire.

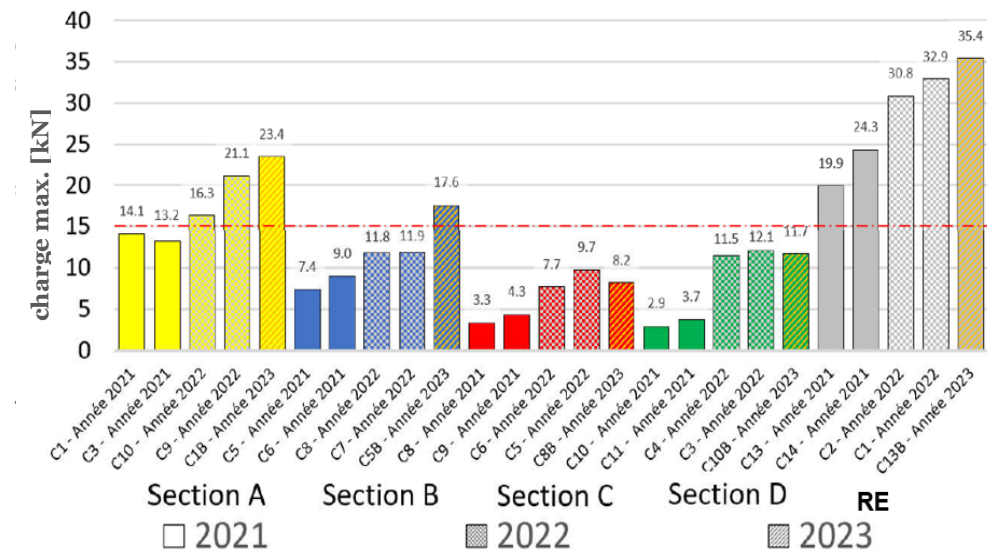


Figure 4 : Résultats Leutner des carottes extraites sur le terrain – Evolution 2021/2022 /2023 (Figure 73 dans le rapport)

Propagation des fissures MMLS3

Les essais de propagation des fissures effectués avec le simulateur de charge de trafic MMLS3 ont révélé que le système A surpasse tous les autres, tandis que le système D est le moins performant, ne supportant qu'environ 60 % des charges que le système A est en mesure de supporter. Les mesures du capteur de déformation (LVDT) ont montré que le système A confère une grande rigidité à la structure, tandis que le système D est le plus faible (voir figure 5).

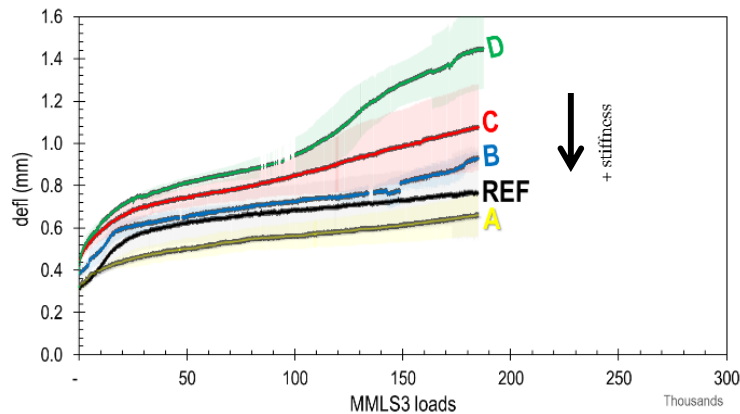


Figure 5 : Pour chaque système, l'amplitude moyenne de déviation en fonction du n° des charges MMLS3 (Figure 57 dans le rapport)

Trois mécanismes distincts de formation et de propagation des fissures ont été identifiés, comme le montre la Figure 6 du rapport :

- 1) Pour certains échantillons, immédiatement après le début du chargement, l'analyse de Corrélation d'Images Numériques (DIC) révèle un décollement des couches, suivi de l'initiation d'une fissure à la pointe de l'entaille, qui se propage à travers l'échantillon jusqu'à atteindre la surface supérieure. Ce mécanisme a été observé dans tous les échantillons du système D, ainsi que dans deux échantillons du système C.

2) Dans le second mécanisme, une fissure verticale commence au niveau de l'entaille, se propage directement à l'interface, provoquant le décollement de la couche, puis continue jusqu'au sommet. Cela suggère que le renforcement est légèrement plus résistant que dans le premier mécanisme. Dans les deux cas, la fissure verticale peut soit se poursuivre directement dans la couche supérieure, soit se propager horizontalement. Un déplacement horizontal indique une intercouche très faible, entraînant un comportement indépendant des couches sous chargement. Tous les échantillons du système B, un échantillon du système C et deux échantillons de référence ont suivi ce mécanisme de dommages.

3) Le troisième mécanisme implique une fissure verticale se propageant directement dans la couche supérieure sans provoquer de séparation de l'interface. Dans les deux premiers cas, la propagation des fissures s'est généralement produite plus rapidement que dans les échantillons où aucune séparation entre les couches n'a été observée. Ce mécanisme de fissuration a été détecté dans tous les échantillons du système A, ainsi que dans 1 échantillon de référence.

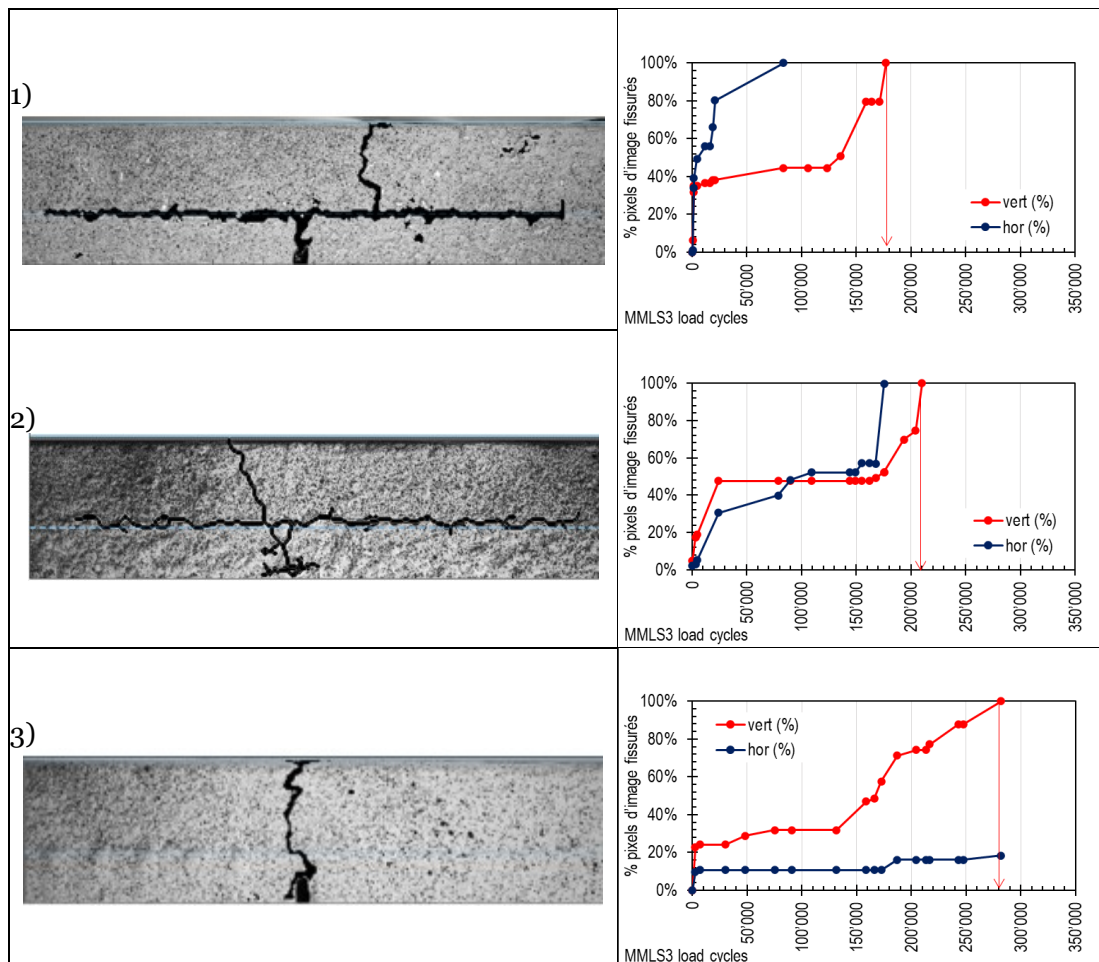


Figure 6 : Trois mécanismes de formation et de propagation des fissures identifiés par l'analyse CID (Figure 63 dans le rapport)

La figure 7 présente le résumé des résultats de la propagation des fissures en termes de cycles de charge avec MMLS3. Le type de rupture est fortement corrélé aux performances du composite testé : les dalles présentant un décollement intercouche échouent

généralement après un nombre de cycles de chargement inférieur par rapport à celles avec des liaisons intercouches intactes.

En résumé, les tests ont démontré qu'un système de grille peut retarder efficacement la formation et la propagation des fissures, à condition d'assurer un collage solide avec le reste de la structure. Cela se traduit par un effet de rigidification de l'ensemble du système. Par conséquent, tous les systèmes de renforcement, à l'exception du système D, sont plus performants que la référence non renforcée.

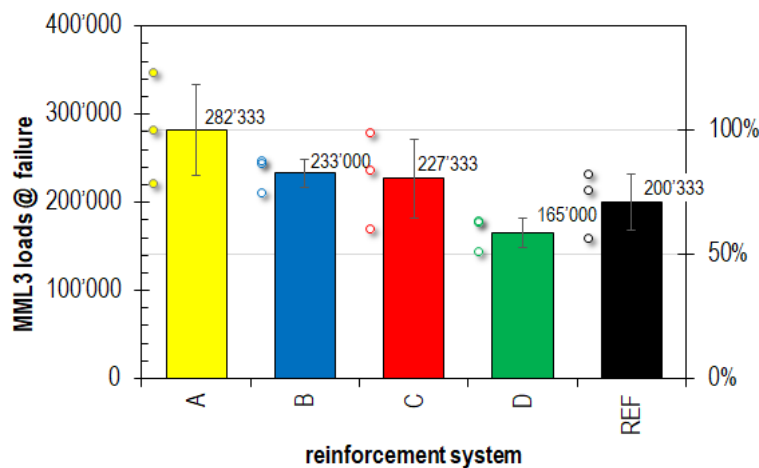


Figure 7 : Nombre approximatif de charges jusqu'à la rupture des dalles (Figure 64 dans le rapport)

Propagation des fissures – Essais de flexion 4 points

Des échantillons de poutres ont été prélevés sur des composites à double couche pour une analyse sur la base de l'essai de propagation des fissures. Les résultats de l'essai de propagation de fissures par poutre de flexion 4 points (4PB) présentés à la figure 8 ont révélé que l'ensemble des éprouvettes renforcées supportaient un nombre plus élevé de charges cycliques, indépendamment du type de grille, par rapport aux éprouvettes de référence non renforcées. Les éprouvettes avec grille de renforcement B semblent être les plus performantes et offrent la résistance la plus élevée contre la propagation des fissures, suivies par la grille D.

Pour évaluer les effets des joints de renforcement, tels qu'ils apparaissent sur le chantier en raison de la pose (par exemple, les joints longitudinaux et transversaux relatifs au recouvrement des grilles de renforcement), l'essai de longueur de recouvrement a été effectué sur des éprouvettes préparées pour l'essai de propagation des fissures à basse température. La longueur de recouvrement longitudinal des grilles de renforcement a été supposée dans les échantillons préparés en laboratoire et soumis à des essais de propagation de fissures sous charge cyclique. L'ensemble des échantillons ont été préparés avec une longueur de recouvrement de 15 cm de la grille de renforcement.

Les résultats des essais ont révélé que les éprouvettes préparées avec grille de renforcement B supportaient un nombre plus élevé de cycles contre la propagation des fissures et présentaient même une résistance plus élevée par rapport à la variante sans recouvrement. Ces résultats indiquent que la longueur de recouvrement offre une résistance plus élevée en utilisant l'essai de propagation des fissures. Cet effet a été ob-

servé pour l'ensemble des éprouvettes renforcées, à l'exception du produit D. Le classement, tel qu'il ressort des résultats des échantillons préparés avec des grilles de pleine longueur, s'est amélioré après l'essai de l'échantillon avec recouvrement des grilles. Il convient de mentionner que le nombre de cycles atteint pour l'échantillon avec grille D et longueur de recouvrement est encore plus faible par rapport aux échantillons de référence. Cela pourrait être lié à la spécificité de l'interface entre la grille D et l'enrobé ; ceci car dans les résultats avec l'essai AST, il a été constaté que les échantillons préparés avec le type de grille D présentaient des problèmes intercouches dans les cas de conditionnement sec et humide. De plus, le type de grille D a obtenu les valeurs les plus faibles lors des essais AST.

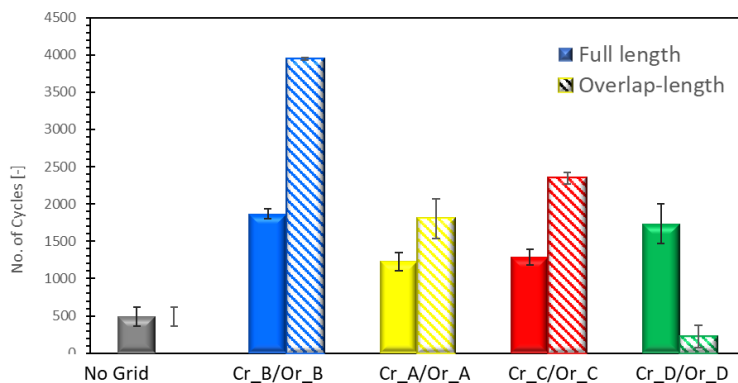


Figure 8 : Comparaison d'échantillons de grilles renforcées (longueur totale et longueur de recouvrement) en termes de nombre total de cycles (Figure 41 dans le rapport)

Section d'essai

La construction d'une planche d'essais sur la T9 à Sierre (canton du Valais) située à la route d'Escala, du kilomètre PR580+740m au PR590+040m a été réalisée le 15.10.2021. Parallèlement aux essais en laboratoire, la réalisation des planches d'essais sur le terrain a été initiée en collaboration avec les autorités cantonales (Service de la Mobilité) et les différents fournisseurs de grilles de renforcement. Les principaux fournisseurs de grilles de renforcement ont donc été invités à participer à la construction des planches d'essais, ceci en posant eux-mêmes leurs propres grilles de renforcement. Cinq différentes sections d'essai soit les sections A à D avec grilles de renforcement ainsi qu'une section de référence sans grille ont été construites. Différentes campagnes de mesures ont été effectuées avant et après la construction de la planche d'essais, à intervalles annuels. Chaque campagne de mesure comprenait l'extraction de carottes pour l'essai d'adhérence intercouche selon Leutner, la réalisation de mesures de portance avec déflectomètre à masse tombante (FWD), un relevé des dégradations de surface avec ARAN et, dans certains cas, une inspection visuelle.

Les indices d'état I1, I2 et I3 présentés dans les Tableaux 2, 3 et 4, ainsi que les valeurs de macro-texture, ont été enregistrées à l'aide du véhicule de mesure ARAN. L'indice de détérioration de surface I1 a montré, pour toutes les sections testées, qu'elles soient renforcées ou non, un mélange de conditions bonnes et moyennes, conformément à la norme suisse VSS40925. Les résultats de l'indice de planéité longitudinale I2 ont révélé pour la section de référence des tronçons dans un état critique ainsi qu'un mélange de zones en bon et moyen état. Pour la section B, une petite partie dans une direction a

été identifiée comme étant en état suffisant, tandis que le reste de la section présentait un mélange de bonnes et moyennes notes. L'indice de planéité transversale I3 a montré que la section de référence et la section B étaient pour la grande majorité en bon état, tandis que les sections A, C et D étaient généralement dans un état moyen.

Les mesures avec un déflectomètre à masse tombante FWD ont été effectuées, et les résultats ont été recalculés à une température de référence de 15 °C. Les modules élastiques recalculés de la couche d'enrobé bitumineux, comme indiqué à la figure 9, ont montré que la section non renforcée présentait des valeurs légèrement plus élevées par rapport aux sections renforcées. Cette différence pourrait être attribuée à la présence de la grille de renforcement à l'interface, ce qui pourrait affecter la répartition des contraintes. Parmi toutes les sections avec des grilles de renforcement, la couche d'enrobé bitumineux avec le Produit A a montré des valeurs de modules légèrement plus élevées que les couches d'enrobé bitumineux avec les autres systèmes de renforcement.

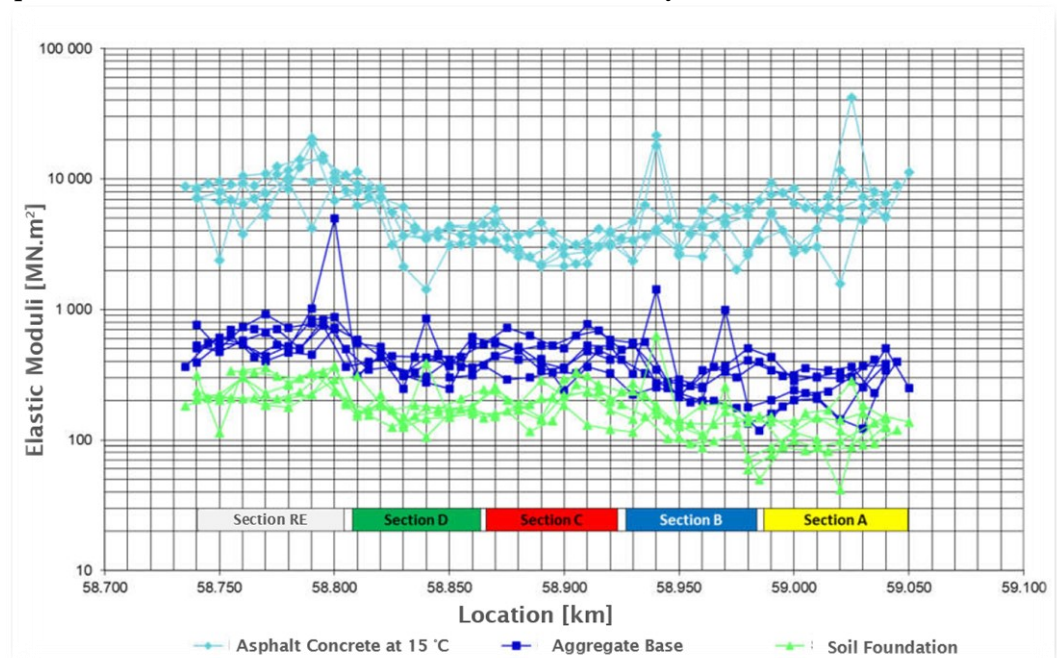


Figure 9: Modules élastiques recalculés de la section de test (Figure 80 dans le rapport)

Dégradation de surface Indice I₁

Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Moyenne	0.9	1.1	1.4	1.2	1.4	1.1	0.9	1.3	1.3	1.1
Minimum	0.8	0.8	1.4	0.8	1.4	0.8	0.8	0.8	0.8	0.8
Maximum	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4

Table 2: Résultats détaillés de l'indice de dégradation de surface I1 (Table A8 en Annexe)

Indice longitudinal I_2

Section	Section RE		Section D		Section C		Section B		Section A	
	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Moyenne	1.1	1.0	1.0	0.5	0.8	0.9	2.2	1.4	1.0	0.7
Minimum	0.3	0.4	0.5	0.0	0.3	0.2	1.5	0.2	0.6	0.4
Maximum	1.6	1.4	1.5	1.0	1.8	2.1	3.2	2.2	1.5	1.0

Table 3: Résultats détaillés de l'indice longitudinal I_2 (Tableau A9 dans l'annexe)Indice transversal I_3

Section	Section RE		Section D		Section C		Section B		Section A	
	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Moyenne	0.8	1.0	1.4	1.2	1.3	1.2	0.6	1.2	1.3	0.9
Minimum	0.5	0.7	1.0	0.6	1.1	0.8	0.3	0.9	1.0	0.3
Maximum	1.1	1.4	1.6	1.5	1.5	1.7	1.3	1.5	1.5	1.5

Table 4: Résultats détaillés de l'indice transversal I_3 (Table A11 in Annex)

De plus, l'état de surface caractérisé par la fissuration a été évalué lors de la campagne de mesure de 2023. L'évaluation a montré que les fissures dans la section de référence étaient principalement dans le sens transversal, tandis que les fissures dans les sections renforcées étaient principalement dans le sens longitudinal. Dans l'ensemble, le nombre de fissures visibles était plus élevé dans la section de référence que dans les sections renforcées. Pour une évaluation plus précise de la fissuration, les données ARAN ont été traitées à l'aide d'une technique d'imagerie. Dans cette évaluation, le nombre de fissures a été comparé et évalué sur la base du nombre de fissures présentes avant la construction de la planche d'essais. Il est à noter que la comparaison a été effectuée sur la base des données de pré-construction disponibles en 2019 dans un seul sens (c'est-à-dire direction Sierre). Les différentes barres de la figure 10 montrent le nombre de pixels sur la base des mesures ARAN en 2019 (avant) et en 2023, soit deux ans après la construction de la planche d'essais. Comme on peut le voir dans la figure 10, le nombre de fissures en 2019 (basé sur le nombre de pixels) est beaucoup plus élevé dans la section de référence par rapport aux sections avec grilles de renforcement. Cependant, les mesures réalisées en 2023 montrent que le pourcentage de fissuration après 2 ans est encore plus élevé dans la section de référence par rapport aux sections renforcées.

La figure 11 indique le nombre de fissures en 2023, exprimées en pourcentage du nombre de fissures présentes en 2019, et ceci pour l'ensemble des sections. Comme mentionné précédemment, le pourcentage le plus élevé (environ 61 %) a été trouvé pour la section de référence, suivie de la section B avec un pourcentage de 50 %. La section A présente le plus faible nombre de fissures par rapport au nombre de fissures avant la construction (environ 9 %), tandis que le pourcentage pour la section C est d'environ 25 %, et de 33 % pour la section D. Le pourcentage de 9 % pour la section A ne comprend pas le joint de pose qui est réapparu en surface et n'est donc pas lié à la performance propre de la chaussée.

Étant donné que le nombre de fissures réfléchissantes (remontée de fissures) est un indicateur pertinent pour l'évaluation de la performance des grilles, le pourcentage de

fissures réfléchissantes (en comparant le nombre de fissures avant et après la construction) a été évalué pour chaque section comme illustré dans la figure 12. A nouveau, la section de référence sans grille de renforcement présente le plus grand nombre de fissures réfléchissantes (43 %), suivie de la section B avec environ 20 %. Dans la section A, le nombre de fissures réfléchissantes est d'environ 2 % (sans compter le joint de pose, qui n'est pas considéré comme une fissure réfléchissante), tandis que dans les sections C et D, aucune fissure réfléchissante n'a été détectée.

Les investigations ont révélé que les grilles de renforcement installées sont tout à fait capables de prévenir ou du moins de retarder l'apparition de fissures réfléchissantes. Cependant, il est difficile de faire une comparaison entre les grilles de renforcement installées dans les différentes sections, car une inspection réalisée en mai 2024 a révélé des structures géométriques très différentes, telles que des courbes et des jonctions, ce qui entraîne des charges de trafic différentes pour chaque section. Dans ce contexte, il est important de préciser la présence du rond-point au début de la section A et la voie de sortie dans la section B.

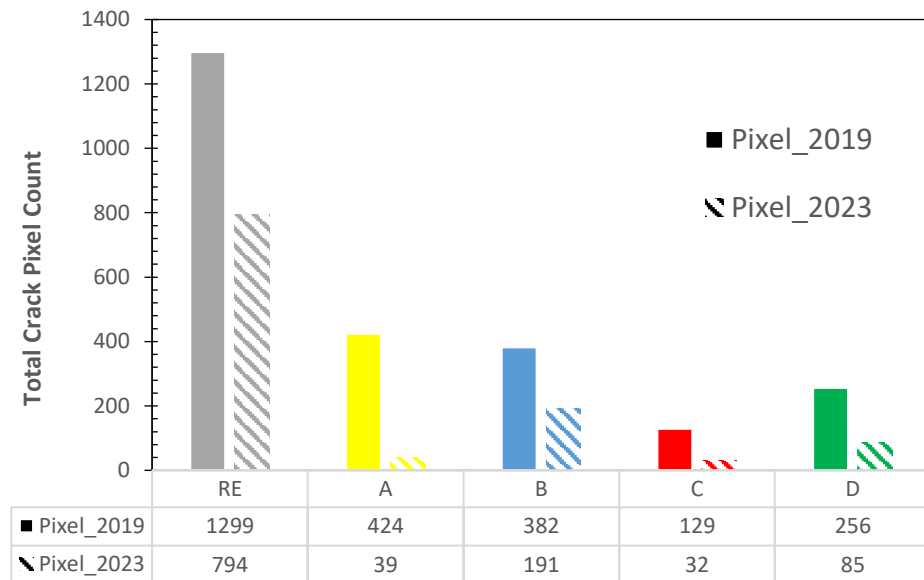


Figure 10: Taux de fissuration de surface mesurée à l'aide d'une technique d'imagerie (pixels) (Figure 88 dans le rapport)

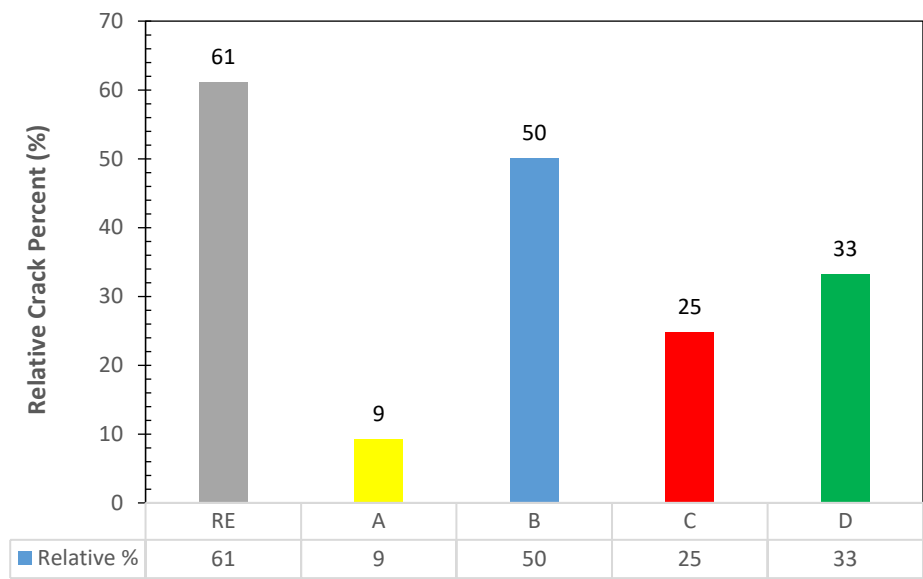


Figure 11: Taux de fissuration de surface en pourcentage relatif (Figure 89 dans le rapport)

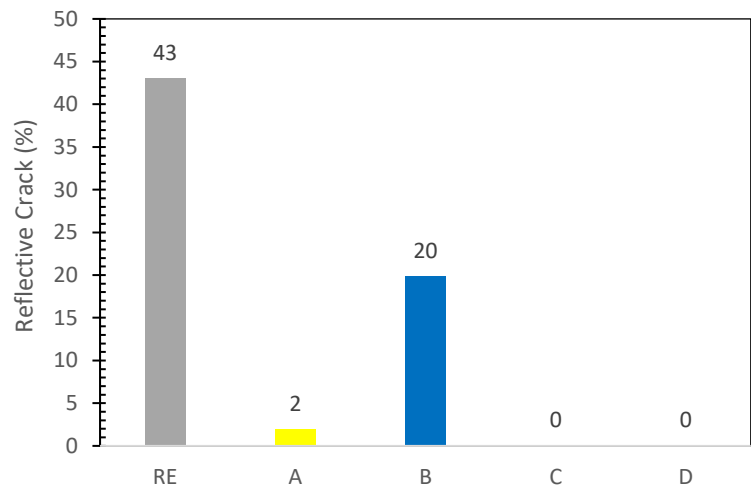


Figure 12: Taux de fissuration réfléchissante en pourcentage relatif (Figure 90 dans le rapport)

Modélisation

Les travaux de modélisation comprenaient l'essai Leutner et l'essai MMLS3, dans le but d'établir les bases pour la prise en compte des grilles de renforcement dans le processus de dimensionnement des chaussées. Pour ce faire, la méthode des éléments finis (FEM) a été choisie comme l'outil le plus approprié pour modéliser la géométrie complexe des grilles et l'interaction entre celles-ci et la structure environnante. Le modèle d'éléments finis utilisé dans ce projet était basé sur le code commercial d'Abaqus. Dans un premier temps, un modèle de l'essai Leutner a été préparé à l'aide de l'environnement Abaqus/CAE afin d'établir le modèle de liaison intercouche le plus approprié et

ses paramètres. Après avoir essayé différents modèles de liaison intercouches pour l'interface, des éléments cohésifs ont été sélectionnés pour simuler l'interaction entre les couches bitumineuses. Le côté gauche de la figure 13 montre la géométrie du modèle et sur la droite, la comparaison des résultats du modèle avec la mesure est présentée. Si les paramètres sont correctement choisis, le modèle peut prédire avec précision l'essai de laboratoire.

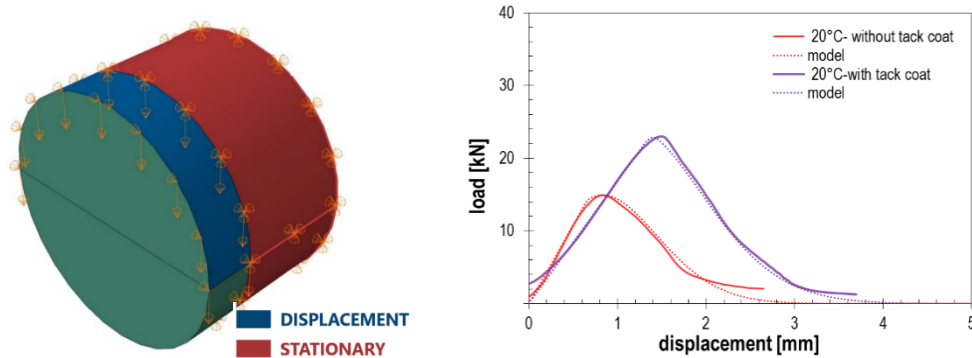


Figure 13 : Géométrie FEM de l'essai Leutner (à gauche) et comparaison des courbes de déplacement de charge obtenues avec l'essai et le modèle (Figure 98 dans le rapport)

Les simulations du test MMLS3 ont été préparées à l'aide d'un plug-in Abaqus programmé en Python. Le plug-in présente une interface utilisateur guidée (GUI) dans l'environnement IAO d'Abaqus (Finite Element Software). Dans ce formulaire, l'utilisateur peut entrer l'ensemble des paramètres des simulations, y compris la géométrie des échantillons et des grilles, les matériaux, la charge, les propriétés d'interface entre les couches et les paramètres de maillage. Avec cette procédure, il est possible de créer différentes simulations 3D avancées à l'aide de charges mobiles, de matériaux bitumineux viscoélastique dépendant de la température, de grilles de renforcement intégrées de différentes géométries et de différents paramètres de liaison intercouche, y compris les éléments cohésifs avec des paramètres calibrés à partir des modèles Leutner précédents (voir figure 14). Le modèle a été calibré à l'aide des déplacements mesurés, ceci pour obtenir un bon ajustement. Comme le montre la figure 15, les résultats comparant les déformations pour la situation non renforcée et la situation renforcée par la grille ont montré une légère différence (voir figure 16). À partir de ce point, le modèle peut alors être extrapolé à un modèle de chaussée réel et différentes grilles de renforcement peuvent être évaluées sans qu'il soit nécessaire de procéder à des essais additionnels.

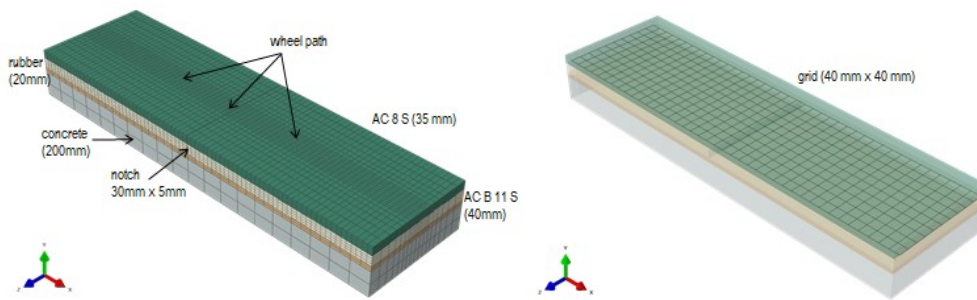


Figure 14 : Vue de la géométrie et du maillage du modèle (Figure 110 dans le rapport)

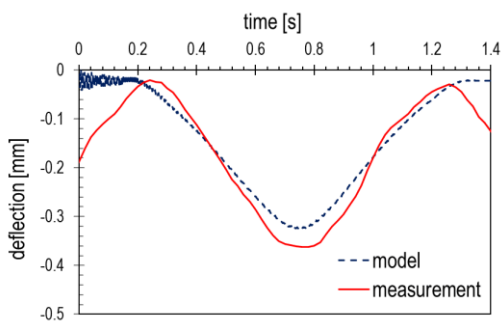


Figure 15 : Calibrage du modèle (Figure 114 dans le rapport)

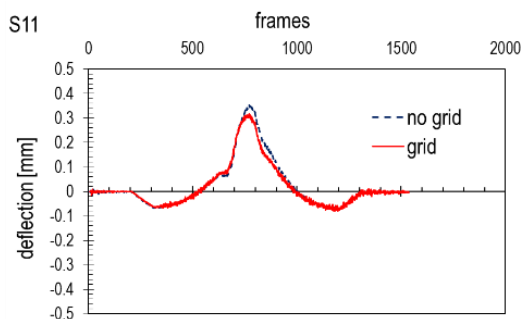


Figure 16 : Déformations dans le sens longitudinal d'un élément au-dessus de l'encoche, dans un modèle avec et sans grille (Figure 116 dans le rapport)

Les résultats de validation confirment la précision du modèle élaboré, offrant ainsi une base pour des analyses ultérieures qui intègrent des propriétés spécifiques de la grille, comme l'espacement des fibres, le module du matériau et la section transversale. Des essais Leutner à différentes températures devraient être inclus dans les travaux à venir pour améliorer les paramètres de liaison entre les différentes couches et les valeurs des éléments cohésifs. Le modèle aura ainsi la capacité d'évaluer les conséquences de la position de la grille et des conditions telles que la charge, la vitesse et la température sur les contraintes et les ajustements du revêtement. L'extension des simulations aux chaussées en vraie grandeur soumises à des charges de trafic permettra de générer des données essentielles pour la conception de chaussées renforcées en utilisant des méthodes mécanico-empiriques.

Analyse du coût du cycle de vie

Sur la base des résultats des lots de travail précédents, une analyse des coûts du cycle de vie (LCCA) a été réalisée. La LCCA repose sur un modèle théorique de durée de vie basé sur des indicateurs clés de performance (KPI), adapté à trois scénarios différents selon les résultats des tests disponibles dans le cadre du projet. Les essais de charge MMLS3 ont été jugés pertinents en raison de leur configuration réaliste. La situation initiale de cette analyse était qu'une chaussée bitumineuse existante présentait des fissures dans la couche de surface. Les couches d'asphalte sous-jacentes avaient déjà atteint la majeure partie de leur durée de vie théorique prévue. La question clé était de déterminer si une reconstruction complète (remplacement de toutes les couches d'asphalte) ou une approche de réhabilitation ciblée — consistant à remplacer uniquement la couche de surface tout en incorporant un renforcement en asphalte — serait plus efficace.

Pour évaluer cela, trois scénarios différents ont été sélectionnés et leurs performances comparées en utilisant des KPI définis. La variante de référence (Ref) comprend le remplacement de toutes les couches d'asphalte au début de la période d'analyse. Les variantes V1 et V2 consistent à remplacer uniquement la couche de surface tout en intégrant un renforcement en asphalte entre la couche de surface et la couche de liaison. Cette approche vise à retarder le besoin d'un remplacement complet des trois couches d'asphalte. Ces deux variantes, V1 et V2, dérivent des résultats des essais de charge MMLS3 et empêchent la qualité existante des chaussées en combinaison avec le système de renforcement en asphalte utilisé.

La variante V1 suppose que les conditions optimales ont été réunies lors du pavage, conduisant à une qualité de pavage maximale. Cela signifie que la liaison inter-couche entre la grille et les couches d'asphalte a pu être réalisée de manière optimale. La variante V2, en revanche, fait partie du principe qu'une liaison optimale entre la grille d'asphalte et les couches d'asphalte n'a pas pu être obtenue. En règle générale, cela implique que les charges de trafic appliquées sur les couches de surface en asphalte ne peuvent pas être transférées de manière optimale aux couches sous-jacentes, entraînant une fatigue prématurée de la couche de surface en asphalte.

L'avantage de chaque variante a été calculé pour chaque KPI et quantifié à l'aide de la somme des aires sous les courbes KPI. Les coûts correspondants comprennent les coûts purement liés à la mesure. Les trois variantes de modèle présentent des avantages similaires en termes de performance des quatre KPI. La variante de référence (Ref) a atteint une superficie totale de 869,5 unités, la variante V1 une superficie totale de 867,9 unités, et la variante V2 une superficie totale de 866,6 unités. Cependant, il existe une différence significative entre les trois variantes en termes de coûts totaux. La variante V2, avec des coûts totaux de 208 687 CHF, est l'option la moins favorable, car elle nécessite une reconstruction complète supplémentaire (E2) au cours de la période d'analyse. En revanche, la variante de référence (Ref), avec des coûts totaux de 165 672 CHF, et la variante V1, avec des coûts de 147 965 CHF, ont généré des coûts nettement inférieurs.

Les résultats de cet exemple d'application montrent que l'utilisation de grilles de renforcement peut apporter un avantage supplémentaire sur une période d'analyse à long terme de 50 ans.

Cependant, si la grille de renforcement n'est pas installée de manière optimale ou ne remplit pas les exigences minimales, comme le collage des couches, une mesure d'entretien supplémentaire peut être nécessaire, ce qui entraîne une augmentation des coûts au cours du cycle de vie. Cependant, l'utilisation de grilles de renforcement peut présenter un avantage dans le cas d'application où le remplacement techniquement optimal de l'ensemble des couches bitumineuses n'est temporairement pas réalisable pour des raisons de circulation, économiques, de personnel ou à d'autres facteurs. De plus, l'impact de l'élimination nécessaire des matériaux d'asphalte fraisés, contaminés par les grilles de renforcement, a été étudié. En fonction des Déclarations environnementales de produit spécifique (EPDs) utilisées pour l'évaluation des unités équivalentes de CO₂ et d'autres indicateurs de durabilité, cela peut entraîner une diminution supplémentaire des bénéfices liés à cette approche.

Discussion

Sur la base des résultats obtenus dans le cadre de ce projet, les points de discussion suivants sont soulevés :

- L'essai de liaison intercouche montre qu'il existe des grilles de renforcement qui, en laboratoire, conduisent à un comportement meilleur ou comparable par rapport aux échantillons non renforcés, tandis qu'in situ, en raison des contraintes et des difficultés de construction, les grilles de renforcement semblent toujours créer une interface entre les couches, ce qui entraîne des propriétés de liaison intercouches plus faibles.
- La liaison intercouche in situ s'améliore en général au fil du temps, ce qui est conforme aux études antérieures (Raab et al., 2017, Raab & Partl, 2007). Certains fournisseurs de grilles de renforcement (grilles avec membrane autocollante) revendiquent même ce mécanisme (compactage de la charge du trafic en combinaison avec des températures élevées en été) pour améliorer la performance de leur produit.
- La comparaison entre les essais de propagation des fissures MMLS3 et 4PB ne s'est pas avérée aisée. Cela pourrait être lié à la production d'échantillons en laboratoire et aux différentes échelles d'échantillons utilisées pour chaque type d'essai. De plus, la température d'essai a été un facteur déterminant puisque les échantillons MMLS3 ont été testés à 20 °C, tandis que l'essai 4PB a été effectué à 5 °C.
- Dans l'ensemble, l'essai MMLS3 doit être considéré comme davantage réaliste et comparable à la situation sur le terrain en raison de sa configuration de charge et de la taille et géométrie des éprouvettes, en particulier en ce qui concerne la taille du maillage des produits de renforcement. En outre, il convient de mentionner que l'essai de propagation des fissures 4PB a été effectué en testant uniquement deux échantillons.
- Bien que l'essai de propagation des fissures 4PB ait ses limites, il a été possible de comparer différents produits, car leurs tailles de maille étaient comparables.
- Les échantillons dont la longueur de recouvrement est de 15 cm se sont révélés davantage résistants lors de l'essai 4PB que les échantillons pleine longueur pour le même essai. Cela montre que le recouvrement des grilles d'armature testées selon les recommandations des fournisseurs ne pose *a priori* pas de problème et que la longueur de recouvrement pour un type de grille spécifique pourrait même être optimisée.

- La planche d'essais choisie s'inscrivait dans le cadre des travaux d'entretien programmés du canton du Valais et a été réalisée en collaboration avec les autorités cantonales. Bien qu'il ait été choisi en raison de son trafic élevé, il subsistait quelques différences entre les planches d'essais soit notamment au niveau de la géométrie (courbes), de la quantité et le régime du trafic (nombre de poids lourds, voies de tournant, forces de cisaillement dues au freinage dans les ronds-points), de l'état de la chaussée en place (taux de fissuration). Comme cela a été observé lors d'une inspection en 2024, des sections avec perte de gravillons et un ravinement de la couche superficielle ont été trouvées ; des phénomènes qui ne peuvent pas être attribués à la performance de la grille de renforcement mais révèlent les différentes contraintes de charge des sections.
- En ce qui concerne l'installation des différentes grilles de renforcement, il convient de noter que l'installation du système A a été réalisé tôt le matin sur une couche de liaison déjà durcie, tandis que les autres systèmes (par exemple, les systèmes B et D) ont été installés sur une couche de liaison relativement non durcie. Ceci se vérifie en particulier pour le système B, qui utilise un taux de couche de liaison très élevé. De plus, comme l'a souligné le fabricant d'un système à membrane auto-adhésive, la température requise de la couche superficielle de 140 °C, qu'il considère comme importante pour garantir les propriétés de liaison de son système, n'a pas été atteinte.
- Une comparaison entre les résultats de laboratoire et les résultats sur le terrain n'a pas été possible pour le produit de renforcement D, ceci car un gravillonnage a été utilisé pour l'installation sur le terrain, mais pas en laboratoire.

Conclusions et recommandations

- Comme mentionné ci-dessus, l'essai d'adhésion intercouche a révélé qu'il existe des grilles de renforcement qui, en laboratoire, conduisent à un comportement meilleur ou comparable par rapport aux échantillons non renforcés, tandis qu'in situ, en raison des contraintes et des difficultés de construction, les grilles de renforcement semblent toujours créer une discontinuité entre les couches, ce qui entraîne souvent des propriétés de liaison intercouches plus faibles. Néanmoins, il existe des grilles de renforcement qui peuvent atteindre la valeur minimale requise de 15 kN selon la norme suisse. Pour cette raison, il est très important que les fabricants continuent d'améliorer leurs systèmes en ce qui concerne le collage de couches d'une part et d'optimiser d'autre part l'installation de leurs produits dans la mesure du possible.
- De leur côté, les propriétaires de routes et les entrepreneurs doivent s'assurer que les exigences nécessaires à la construction de chaussées soient strictement respectées lors de l'installation de grilles de renforcement et que toutes les conditions pour une installation optimale des grilles de renforcement spécifiques soient remplies.
- En utilisant différentes méthodes d'essai en laboratoire pour évaluer la propagation des fissures (MMLS3 et 4PB), l'effet de pontage et de retardement de la remontée de fissures des grilles de renforcement et les différences avec les systèmes sans grilles de renforcement ont pu être clairement démontrés pour différents types de grilles. De plus, à cet égard, la liaison intercouche s'est avérée être l'un des facteurs détermi-

nants de la performance d'un système de chaussée renforcée. Pour obtenir de meilleures performances, il est donc primordial d'améliorer la liaison entre les couches, ce qui est également crucial pour prévenir les dommages relatifs à la sensibilité à l'eau.

- Il a été démontré que la combinaison d'essais sur simulateur de trafic à l'échelle du laboratoire (essais MMLS), de mesures de déflexion avec LVDTs et d'une analyse de données de corrélation d'images numériques (DIC) est un outil important pour comprendre le mécanisme des systèmes avec grilles de renforcement, en particulier avec la nouvelle méthode proposée pour déterminer les modèles de propagation des fissures horizontales et verticales. L'évaluation de la propagation horizontale et verticale des fissures démontre que la performance des grilles de renforcement dépend de différents mécanismes et qu'il peut être trompeur de se fier qu'à des essais à petite échelle.
- Les résultats des essais avec recouvrement ont montré que le recouvrement des grilles de renforcement, s'il est effectué conformément aux recommandations des fournisseurs, ne pose pas de problème et que la longueur de recouvrement d'un produit spécifique pourrait même être optimisée.
- Lorsque l'on examine les essais effectués, comme mentionné ci-dessus, l'essai de propagation des fissures MMLS3 doit être considéré comme davantage réaliste et comparable à la situation sur le terrain par rapport à l'essai 4PB, en raison de sa configuration de charge et de la géométrie des éprouvettes, en particulier en ce qui concerne la taille du maillage des produits de renforcement. Bien que l'essai de propagation des fissures 4PB ait ses propres limites, il est plus facile à réaliser et utile pour comparer différents produits. Pour augmenter le niveau de confiance des résultats d'essais, il est essentiel de le réaliser en utilisant un plus grand nombre d'échantillons (pas uniquement 2 échantillons).
- Une simulation numérique précise des tests MMLS3 a été développée et validée, démontrant une grande précision dans la prédiction des réponses structurelles. Le modèle intègre des propriétés avancées des matériaux, des conditions de charge, la liaison inter-couche et la géométrie des grilles. Il distingue le comportement des échantillons avec et sans renforcement, bien que des travaux futurs soient nécessaires pour évaluer les différents types de grilles individuellement.
- L'élargissement de cette modélisation permettra d'évaluer les systèmes de renforcement en utilisant les données des fournisseurs, notamment le besoin de tests physiques intensifs et facilitant l'évaluation dans des conditions variables. Cela fournit des éléments essentiels pour la conception mécanico-empirique des chaussées, garantissant un dimensionnement plus précis et efficace des chaussées renforcées.

En examinant la planche d'essais, les évaluations visuelles de l'état de la surface ont montré que le nombre de fissures était plus élevé dans la section de référence par rapport aux sections renforcées. Les fissures dans la section de référence étaient principalement transversales, tandis que les fissures dans les sections renforcées étaient principalement dans le sens longitudinal.

Pour une évaluation plus précise de la fissuration, les données ARAN ont été traitées à l'aide d'une technique d'imagerie. Selon cette évaluation (chaussée direction Sierre), le

nombre de fissures a été comparé et évalué sur la base du nombre de fissures avant la construction de la piste d'essai. Il a alors été constaté que le nombre de fissures avant la construction était déjà beaucoup plus élevé dans la section de référence par rapport aux autres sections renforcées. Cependant, lors du calcul du pourcentage de fissures avant et 3 ans après la construction, le nombre de fissures dans la section de référence était encore supérieur au nombre de fissures dans les sections renforcées.

Pour la section de référence, le pourcentage le plus élevé a été trouvé avec environ 61 %, suivi de la section B avec un pourcentage de 50 %. La section A présentait le plus faible nombre de fissures par rapport au nombre de fissures avant la construction (environ 9 %), tandis que le pourcentage pour les sections C et D était de 25 % et 33 % respectivement. De plus, le pourcentage de 9 % pour la section A ne comprenait pas un joint de construction qui a réapparu à la surface et ne pouvait donc pas être lié à la performance du système global.

Étant donné que le nombre de fissures réfléchissantes (remontée de fissures) est pertinent pour l'évaluation globale de la performance des grilles, le pourcentage de fissures réfléchissantes (en comparant le nombre de fissures avant et après la construction) a été évalué pour chaque section. A nouveau, la section de référence présentait le plus grand nombre de fissures réfléchissantes (43 %), suivie de la section B avec 20 %. Dans la section A, le nombre de fissures réfléchissantes était de 2 %, tandis que pour les sections C et D, aucune fissure réfléchissante n'a pu être trouvée.

Les investigations sur le terrain ont donc révélé que les grilles de renforcement installées sont tout à fait capables de prévenir ou du moins de retarder l'apparition de fissures réfléchissantes.

Cependant, il était difficile de comparer entre elles les grilles de renforcement installées dans les différentes sections, car une inspection en mai 2024 a confirmé des géométries passablement différentes sous forme de courbes et de jonctions et donc des charges de trafic différentes pour les différents tronçons. Dans ce contexte, il convient de définir la présence du rond-point au début de la section A et de la voie de sortie de la section B.

- Toutes les évaluations et tous les tests effectués au cours du projet ont révélé que le nombre d'échantillons de laboratoire et de terrain doivent être augmenté pour pouvoir présenter des résultats fiables et fournir des recommandations de normalisation et d'application.
- La modélisation développée dans le cadre de ce projet de recherche permettra de déterminer les performances de différents systèmes de renforcement à l'aide des données de produit fournies par les fournisseurs grilles de renforcement. Grâce à cet outil, le nombre d'essais a pu être considérablement réduit et différentes conditions d'essais ont pu être facilement évaluées.

1 Project and objectives

The aim of the project was to investigate and promote best practice for the use of reinforcement grids in bituminous pavements by providing the methodological and normative basis necessary to consider them in a design process and to evaluate their effectiveness (performance). The general basis of the project did not refer to a specific product, which is why the main suppliers of reinforcing products operating in Switzerland were considered and why the results of this work are presented anonymously. For this reason, the four different reinforcement products purchased from different suppliers were labelled with the following designation codes:

- Product A
- Product B
- Product C
- Product D

The choice of the grid reinforcement was made based on the previous work carried out by Empa as well as the data from project (Raab et al, 2017). The application of an identical methodology for the analysis of the different grids as well as that of the grid/asphalt composites, will allow to highlight any differences between the products present on the Swiss market and thus determine their optimal area of use.

After taking note of the state of the art and the latest developments, the data collected in the laboratory and in the field should be used to refine the pavement modelling and thus evaluate the advantages in terms of durability and materials. Finally, the accumulated data and the experience from the laboratory and field tests as well as the life cycle assessments will make it possible to propose updates for the standardisation and maintenance of reinforced pavement structures according to the following Swiss standards VSS 40 324, VSS 40 733, VSS, 2008.

2 State of the art

The following chapter contains an overview of the current state of the art regarding the use of asphalt reinforcement in road construction in practice as well as in research. For this purpose, a brief description of the definition and functionality as well as the application range of asphalt reinforcement is given. Furthermore, the results from national and international research projects are summarized.

2.1 Introduction

Asphalt pavements exposed to heavy traffic and weather conditions reach their bearing capacity limits over time and show various types of damage (cracks, ruts, etc.) that affect the comfort and safety of users. The aim is therefore to seek efficient maintenance and repair methods that make it possible to reduce the cost of measures and increase service life, as well as to save materials and energy. With an ageing road network, Switzerland is inevitably facing this problem. According to the Federal Roads Office (FEDRO), around 40 % of the surface courses of the national network were built before 1998 and thus reaching a service life of more than 20 years. The need for maintenance measures is therefore very high.

Diverse types of measures are available for the maintenance of road infrastructure, whereby reinforcement techniques are becoming increasingly important due to the inclusion of reinforcement meshes (FGSV 2013; Nguyen et al. 2013; Al-Qadi et al. 2008). In the field of pavement reinforcement, there are some scientific studies showing positive results on the effects of the service life of pavements reinforced with these materials (Al-Qadi et al. 2008, Button & Lytton 2007, Raab et al. 2016, Saraf et al. 1996, Partl et al. 2008, Taherkhani & Jalali 2017, Kim et al. 2009, Arsenie et al. 2012). In addition, the ASTRA 2011/011 study provided very interesting preliminary results on the use of reinforcement grids and the associated performance improvements (strength and durability) (Raab et al. 2017). For their part, the manufacturers of the grids have carried out research on their materials and have acquired a certain know-how that enables them to advance their solutions and successfully integrate their products on construction sites. However, these studies independently use different methods and tests, and the results obtained are basically only valid for the product in question.

Although asphalt inlays have been used in the bound superstructure or under surface treatments since the early 1980s (FGSV 2013), the use of reinforcement grids has not yet been integrated into the Swiss standardization, despite the obvious need for dimensioning and good practice. The lack of current normative regulations and objective practical experience ultimately means that reinforcements are not optimally utilized, which leads to losses of more than just an economic nature, as is the case with infrastructure operators who contribute to the maintenance of road networks.

2.2 Fields of application

Originally used in the construction of new roadways, asphalt inlays are also used in the field of maintenance, primarily to prevent cracking (e.g., in asphalt pavements on existing concrete superstructures). The aim is to delay the occurrence and propagation of cracks in the pavement structure and thus enable an extension of the service life before the next maintenance measure is carried out. The following line of application of asphalt inlays can be derived from FGSV (2013):

- Extension of maintenance intervals
- Reduction of the maintenance effort
- Extending the useful life of the road

These aspects must be considered by the manufacturer when introducing a new product and the corresponding properties of the materials or the product must be documented.

2.2.1 Requirements

In addition to the requirements relating to the mechanical properties of asphalt paving, there are additional requirements regarding:

- the ageing resistance
- the reuse and
- environmental compatibility

These aspects must be considered by the manufacturer when introducing a new product and the corresponding properties of the materials or the product must be documented.

2.2.2 Construction method and materials

Asphalt inlays are divided into three product groups: Asphalt grids, nonwovens and combinations of nonwovens and asphalt grids (Tazl 2013). The structure results from the arrangement of the fibres or webs and the steel wire in the asphalt inlay. Fibres are produced from the materials glass and carbon, which are further processed into fibre bundles. The fibre bundles are arranged into a specific structure using different process techniques. The arrangement of the fibres results in an asphalt inlay with the following structures:

- latticed
- planar
- latticed and planar

Figure 1 shows some examples of different reinforcement grids.

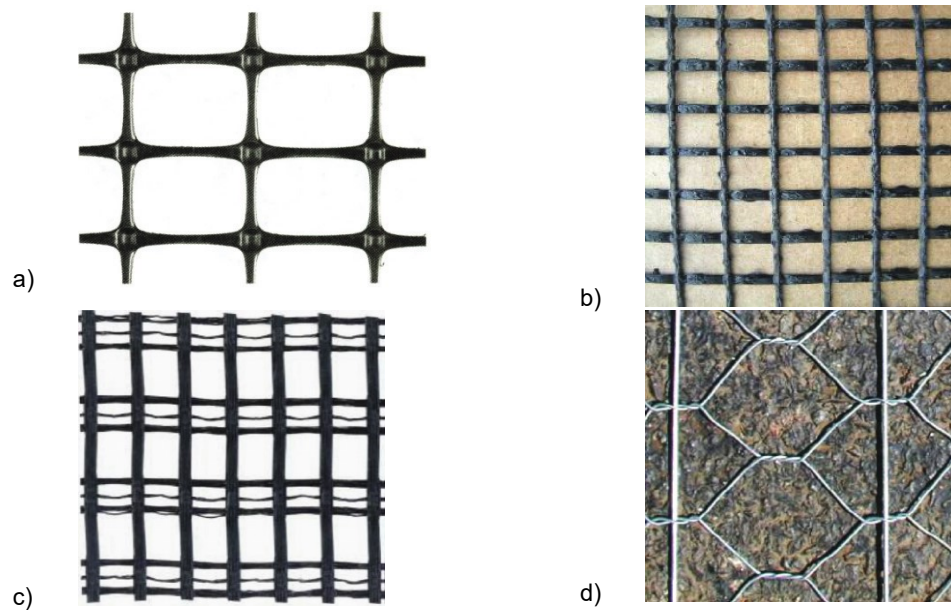


Figure 1: a) Extruded plastic grid, b) Glass fibre grid, c) Polyester grid, d) Steel grid (Rüegger & Hufenus 2003)

2.2.3 Mechanisms

For the use of asphalt inlays, three fundamentally different mechanisms of action must be considered, which in practice do not occur in their pure form, but usually overlap each other (FGSV 2013):

- Sealing effect
- Stress-relieving effect through "flexible bond"
- Reinforcing effect by taking over tensile stresses
- Combined effect

The positive influence of asphalt reinforcement on the service life of pavements can be seen in numerous studies. The possible positive effects include:

- Reduction of lane formation (Guler & Atalay 2016),
- Increased load-bearing capacity of the pavement (Raab et al. 2016, Saraf et al. 1996, Partl et al. 2008),
- Increased resistance to brittle cracking due to cold snaps (Partl et al. 2008),
- Delayed onset or limited speed of reflective crack propagation (Button & Lytton 2007).
- Increased fatigue strength (Partl et al. 2008, Kim et al. 2009, Arsenie et al. 2012),
- Minimisation of differential settlement (Holtz 2017),
- Increased load-bearing capacity of the building platform (Holtz 2017).

Depending on the problem, the reinforcement grid can be laid at various points within the superstructure. For example, in new construction, grids can be placed on the subgrade, on top of the substructure (subgrade or the bound base courses of asphalt mix, unbound or hydraulically bound base courses) to improve the performance of the structure against rutting and fatigue of the material. In other cases where reflective cracking of the material is a problem, the grid can be incorporated into the bituminous layers. In these cases, there is an additional issue to consider; shear strength at the confinement level, knowing that certain types of grids weaken the bond between the

bituminous layers. The contribution of the grid in these cases is mainly to retard the propagation of the crack and to a lesser extent to prevent its occurrence. The preliminary analysis carried out in the preparation of this proposal has highlighted the following aspects that will be considered in the project. Table 1 summarizes the effect of reinforced asphalt layers on the damage characteristics of an asphalt superstructure.

Effect of reinforced asphalt layers on stiffness and cracks

Deflections (stiffness)		Cracks (stability)		
settlements	rutting	thermal	reflection	
+	+	+	+	new asphalt
–	–	–	+	old asphalt
+ good effect / – bad or no effect				

Table 1: Effect of reinforced asphalt layers (Lindenmann et al. 2009)

2.3 National studies

In Switzerland, important findings on the effectiveness and durability of asphalt inlays in the layer system of the road pavement could be obtained within the framework of two research projects (Lindenmann et al. 2009; Raab et al. 2017).

The aim of the VSS 2007/702 research project was to identify and analyse asphalt reinforcement products, the basis for their application, existing research, and practical experience with pavement reinforcement. It was found that pavement reinforcements were made of different materials such as plastic, glass, carbon and steel. These include grid or membrane structures (geotextiles) or a combination of both. The market investigation revealed that at least 18 products were available (12 glass fibre-based products, 3 polypropylene or polyester based products, 2 carbon fibre-based products and 1 steel based product). 26 maintenance projects with integrated pavement reinforcement were identified, of which 23 use either glass fibre or carbon fibre mesh. A detailed analysis of the existing literature at the time was carried out.

Raab et al. (2017) determined the influence of the different reinforcement systems on the pavement behaviour and service life and to determine the optimal position of the mesh in a reinforcement process. This project will serve as the basis for the study proposed here. Laboratory tests with reduced pavement models (using the weight traffic simulator MMLS3) and full-scale tests (using the traffic simulator MLS10) have been carried out. This experimental study is also accompanied by finite element modelling to calculate the influence of the reinforcement grid on the pavement behavior.

Both laboratory and field tests have shown that asphalt reinforcement systems can delay the occurrence of cracks and thus extend the service life of the pavement. These tests also showed that the reinforcement network is a separation layer that can lead to deterioration of the pavement structure if adhesion is poor. Finite element calculations provide an initial indication of the optimal position of the reinforcement grid and can be used as a basis for developing design procedures.

The traffic simulator MMLS3 on pavement models was presented in this study as a relevant test to highlight the effects of reinforcement on fatigue strength. Thus, it was shown that some reinforcement systems can achieve at least twice the fatigue life of the reference system without reinforcement. This test is very sensitive and makes it possible to compare different types of reinforcement in terms of crack propagation resistance.

2.4 International studies

At the European level, the first studies on the use of asphalt reinforcement were carried out as early as the beginning of the 1990s. Figure 2 below gives an overview of the studies carried out in Europe to investigate the use of asphalt reinforcement. These include the publication of guidelines and codes, laboratory investigations, test fields and test tracks. An overview of the most important studies is given in the following subchapters.

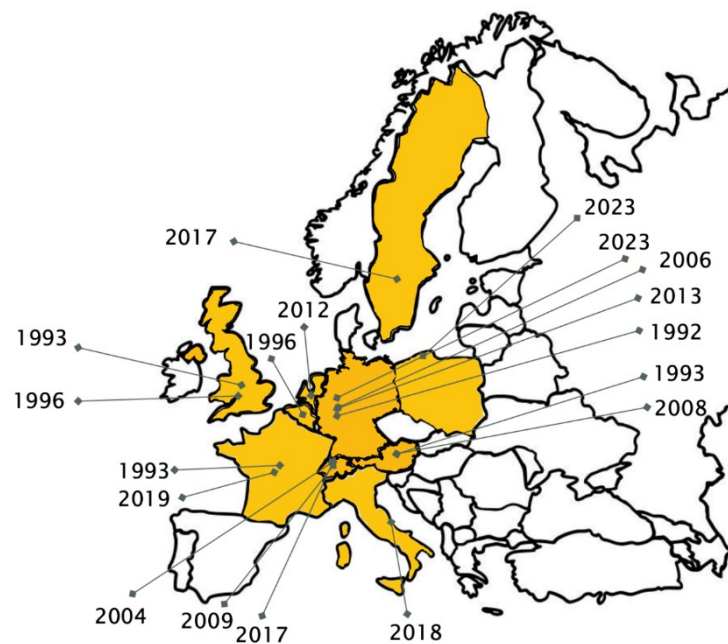


Figure 2: Overview to European studies on the use of asphalt reinforcement

2.4.1 Guidelines and regulations

In the DACH countries, the national standards entitled "Geotextiles and geotextile-related products - Characteristics required for use in the construction of road pavements and asphalt wearing courses" (DIN EN 15381:2008; ÖNORM EN 15381; SN 670259a-NA (2008)) are based on the European standard. The requirements for asphalt inlays according to the Construction Products Ordinance are described in a harmonised European way in DIN EN 15381, which also determines the quality assurance for use. However, for many properties, no concrete requirement values are defined. In Germany, the working paper for the use of nonwovens, grids and composites in asphalt road construction (FGSV 2013) compiles the essential technical principles for the use of asphalt inlays. After the publication of a "Statement on the use

of nonwovens, scrims and grids in asphalt road construction" in 1992, the working paper was first published in 2006 and updated in 2013.

2.4.2 Laboratory tests

Model tests under laboratory conditions are an indispensable tool, as the investigation period can be considerably minimized by time-lapse methods, parameters can be investigated under defined boundary conditions and insights into crack development and propagation can be gained.

Tazl (2013) was able to prove by means of cyclic bending tensile tests that the effectiveness of an asphalt layer depends on the one hand on the applied loads and on the other hand on the material and composite properties. The material properties were determined by means of strip tensile tests, the composite properties by means of interaction tests in the large frame shear tester. It is recommended that the choice of asphalt layer should be directly dependent on the expected load. To quantify the increase in service life due to asphalt layers, additional information on the actual traffic load and the environmental influences should be evaluated and the test parameters optimized to this end.

Mollenhauer et al. (2023) applied different methods (shear tests, permeability tests, 3-point bending tests and wedge splitting tests) to test the stress-relieving or stress-absorbing properties of asphalt overlays. The differences between systems without asphalt reinforcements could be clearly identified for different asphalt products. In order to verify a potential service life prolonging effect, essential parameters were determined by means of an FE model, so that fatigue curves derived in 3-point bending tests, as well as crack energies determined in wedge splitting tests, could be integrated in new verifications against reflection cracking in the computational dimensioning.



Figure 3: 3-point bending test

However, the results showed large differences between the calculated theoretical service lives, which were due both to the underlying assumptions in the applied primary action models and to the test results determined individually on each asphalt inlay system.

Jaskula et al. (2023) also analyzed the effect of multiaxial geocomposite reinforcement on fatigue performance and crack propagation delay in double-layered asphalt beams. The results of these laboratory tests using the 4-point bending test showed that enhancing the pavement with the geocomposite containing multiaxial geogrid increases its fatigue life. The positive effect of geocomposite reinforcement can be quantified by a factor of relative increase in the number of cycles to the first initiation

of crack in lower asphalt layer when reinforcement is introduced that ranges from 5 to 10.

In their study Sudarsanan and Kim (N. Sudarsanan and Y. R. Kim, 2024) quantified the crack resistance capacity of geosynthetic-reinforced asphalt concrete AC using a modified four-point beam fatigue test, namely the Notched Beam Fatigue Test (NBFT). This method evaluates the endurance of reinforced AC to repeated load cycles before failure, providing a tangible measure of crack resistance. The failure criterion in laboratory asphalt fatigue testing is pivotal; the methodology focuses on simple measurements such as load and displacement, deliberately avoiding reliance on visual crack observation and advanced techniques like Digital Image Correlation (DIC). Nevertheless, correlating traditional failure criteria with DIC results were integral to the study, aiming to enhance the predictability of field performance and assist in paving design decisions. A comprehensive explanation of the reflective cracking mechanism, including the implications of different failure modes during testing, was provided.

3 Field test section

3.1 Overview

The construction of a test track T9 Sierre located on Route d'Escala, chainage PR580+740m to PR590+040m in Sierre (canton of Valais) was carried out on 15.10.2021. The test section consists of four different reinforcement grids sections along with the unreinforced section called control or reference section. A detailed summary of the test sections is given in Table 2.

Test section details

Section	Change	Approx. surface area	Code	Product
A	580+985 to 590+040	55 m x 7.5 m	A	Grid Product A
B	580+930 to 580+985	55 m x 8.5 m	B	Grid Product B
C	580+870 to 580+930	55 m x 7.5 m	C	Grid Product C
D	580+810 to 580+870	55 m x 7.5 m	D	Grid Product D
Reference	580+740 to 580+810	55 m x 7.5 m	RE	Reference section without grid

Table 2: Details of test section

Figure 4 to Figure 9 present pictures of the different construction phases. Before construction of the test sections, milling of the old surface layer took place as shown in Figure 4. The dust cleaning as shown in Figure 5 of the milled surface was also carried out. In Figure 6, the application of the emulsion on the cleaned surface after milling is shown. Figure 7 shows the laying of the grid reinforcement during the construction of the test track. The stone chipping specifically used in test track Section D can be seen in Figure 8. Finally, the construction of asphalt surface course is shown in Figure 9.



Figure 4: Milling of the existing surface of the test track



Figure 5: Dust cleaning of the milled surface



Figure 6: Spraying of the emulsion on the milled surface



Figure 7: Laying of the grid reinforcement



Figure 8: Stone chipping on one of the reinforcement sections



Figure 9: Placement of asphalt surface course

The general information of the test track construction such as section length, details of grid type, overlap length, emulsion and its dosage, chipping etc. according to the corresponding grid reinforcement suppliers is provided in Annex 11.4, Table A.

The experimental on-site measurement campaign was carried out as described below:

- Before the construction: characterization of the test section in terms of bearing capacity.
- During construction: characterization of roughness and cleanliness of the test section.
- After construction: characterization of the surface condition and establishment of the initial condition indices.
- Over the years: monitoring of pavement bearing capacity and condition indices at different stages; interlayer bonding (Leutner)

In the past three years the following measurements were carried out:

- Extraction of cores (2 cores per section, i.e. in total: 10 cores)
- Investigation on cores: visual inspection, layer thickness, Leutner test
- Recording of condition indices I_1 , I_2 and I_3 as well as Average Texture Depth (PMT).

- Macrotexture values (PMP) using the ARAN vehicle.
- The FWD measurements were carried out at a rate of one point every ten meters per band and per direction of travel.
- The plan of the test track as shown in Figure 10. It is divided into four different sections i.e. A, B, C, D & RE comprising the four different types of reinforcement grids and the unreinforced reference section.



Figure 10: T9 Sierra – Test track with different sections

3.2 Measuring campaigns

The field measurements campaigns were conducted during the years 2021, 2022, 2023. The ARAN survey along with visual inspections, dust measurement and mean texture depth (during construction phase) and core extraction was carried out during each measurement campaign. It is noted that a backup measurement campaign was planned in 2024, and the results of the measurements are in process and not included in this report.

3.2.1 Field cores

In summer 2021 two cores for Leutner testing were taken from each section. These cores were extracted in the same axis with an offset of about 1m from the cores taken directly after construction. The location of the extracted cores is shown in Figure 11. The table with the cores thicknesses is given in Figure A1 of the Annex § 11.4.

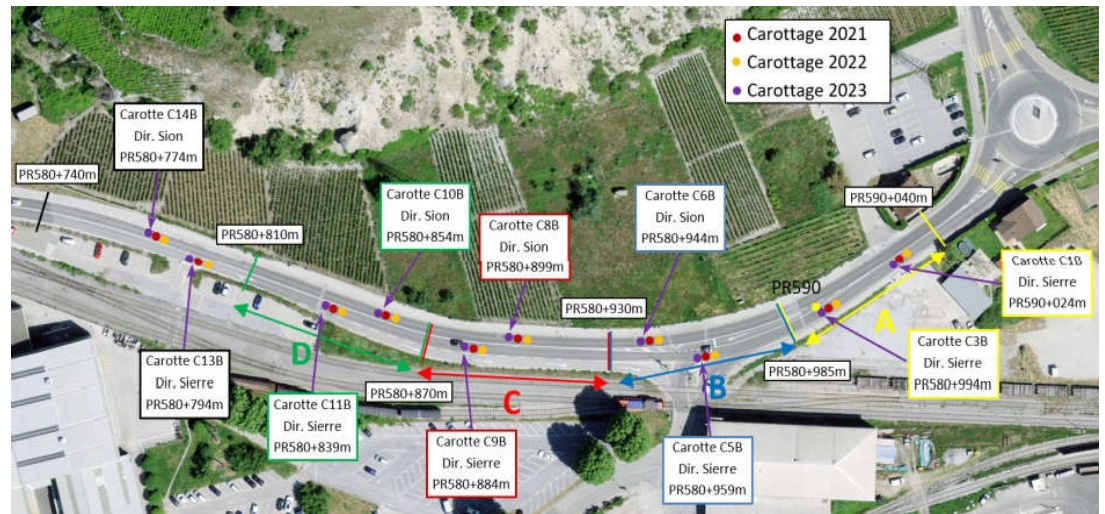


Figure 11: T9 Sierre – Extraction of core drilling in years 2021, 2022 & 2023

The Leutner test results are visible in Figure 12. The results show that the layer adhesion of the reference section has the highest value in comparison to all reinforced sections irrespective of the reinforcement type used and therefore the best interlayer bond properties. Regarding the reinforcement systems, the product used in Section A has the higher values followed by System B. The reinforcement systems in sections C and D never seem to be able to reach the standard requirement of 15 kN. From the results it is also clear that the layer adhesion condition improved over time, a fact that is in accordance with earlier investigations (Raab et al., 2017, Raab & Partl, 2007). The increase for systems A, B and the reference are in the order of 40 %, while the increase for System C and D is 66 % and 76 % respectively.

System A could proof its superior bonding qualities in the laboratory, it is worth noting that its installation was conducted early in the morning on already cured tack coat, while other systems (e.g. system B and D) were installed on a relatively uncured tag coat, which therefore affected the bonding quality. This is especially true for System B which uses a very high tack coat rate.

Further, as highlighted by the producer of System C, the required temperature of the surface course of 160 °C, which is important to ensure the bonding properties of the system, was not achieved (see protocol in Annex § 11.5).

Comparing the field results with the laboratory Leutner test the following can be said: Although only 2 field cores were tested for each reinforcement system, the ranking appears to be similar (System A best, followed by System B). Like in earlier investigations (Raab et al., 2017) there are systems that show in the laboratory a better or comparable behavior to unreinforced specimens, while in situ, due to construction constrains and difficulties, the reinforcement grids always provide a discontinuity between layers, often leading to lower interlayer bond properties.

The poorer performance of systems C and D was also found in the lab testing, while it is worth noting that System D tested on site (with chipping) was different to System D tested in the lab.

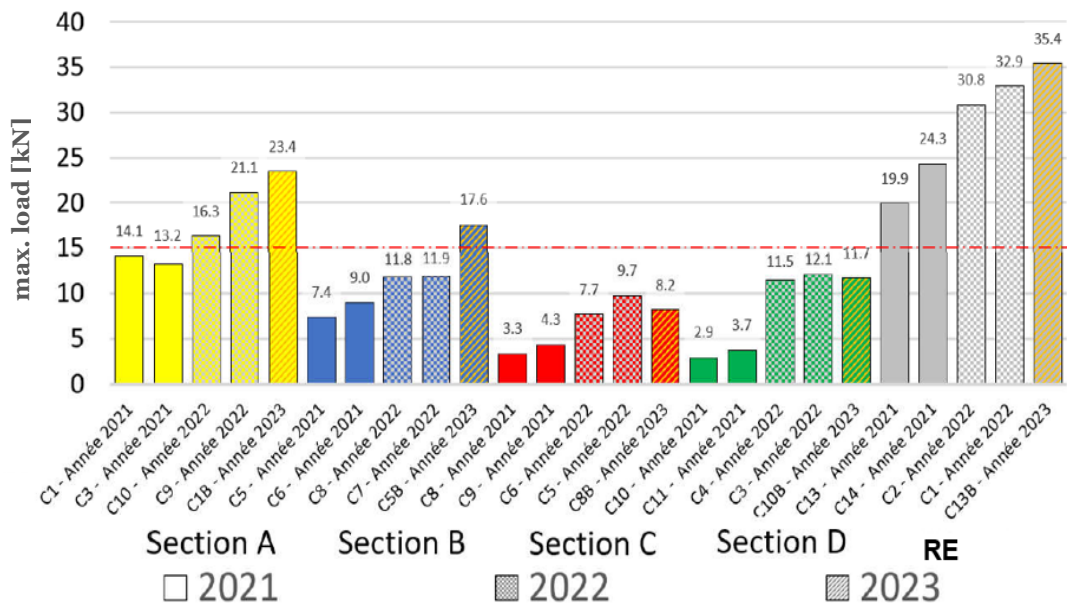


Figure 12: Field extracted cores Leutner results – Evolution 2021 / 2022 / 2023

3.2.2 Condition index survey

The initial condition indices I_1 , I_2 and I_3 and the macrotexture were recorded using the ARAN measuring vehicle as shown in Figure 13. The measurement system is described below.

The ARAN (Automatic Road Analyzer) is a multifunctional vehicle that simultaneously allows:

- Measure longitudinal evenness, transverse evenness, and macrotexture at any point on the pavement using LCMS2 sensors
- Measure road geometry (superelevation, slope, radius of curvature) using a positioning and Geometry System (POSLV)
- Capture 3D images of the pavement using LCMS2 sensors
- Capture 360° forward, backward and panoramic images using surrounding cameras and 360°

All readings and measurements are georeferenced and can be carried out at speeds without hindrance to traffic.



Figure 13: ARAN Measuring Vehicle

Surface degradation - Index I1

The summary survey of surface degradation is carried out using the high-resolution images of ARAN and according to VSS 40 925b, considering the following groups of distresses shown in Table 3.

- Surface slickness
- Surface defects
- Surface deformations
- Structural distresses
- Patches

Classification of road distress

Main groups	Distresses
Surface slickness (skidding hazards)	Polished aggregate Bleeding
Surface defects	Fretting Weathering (surface wear) Raveling Popp outs Potholes Open joints Transverse cracking Single cracks
Surface deformation	Rutting Bumps Corrugation Shoving
Structural distresses	Cracks from depression Depression Lane shoulder drop-off Longitudinal cracking Alligator cracking Edge cracking
Patches	Patches

Table 3: Distress group classification

Measurement of condition indices I_1 , I_2 and I_3 as well as the macro texture values were recorded using the ARAN measuring vehicle. The measurements were carried out at allowable speeds, thus without hindrance to traffic.

Longitudinal Flatness of the Pavement – Index I2

The longitudinal profiles are measured with a measuring step of 1 mm and operated with a measuring step of 10 mm. The W and s_W values are calculated according to VSS 40 517. The W value represents a change in slope. The W value is defined in the real

profile as the angle formed by two adjacent strings 1 m in length. It is used for the assessment of local geometric irregularities and for the determination of the s_w value. Every 10 mm, a measuring point is recorded by the ARAN system.

The s_w -value represents the standard deviation of the W-values for a 50 or 100 m long reach. In contrast to the W value, it is used for the overall assessment of larger sections. For this project and for a more detailed assessment of the longitudinal flatness, the s_w values are calculated with a step of 10m. The index value I_2 of VSS 40 925b is calculated from the value s_w using the curves for priority roads RP. The I_2 index is assessed according to the classification (road class HVS) as shown in Annex § 11.4.

Transverse Flatness of the Pavement – Index I_3

The transverse profiles are measured with a measuring step of 1 mm and are operated with a measuring step of 10 cm. From these profiles, the flangeway depths T are calculated according to the chord method of the VSS 40 518 standard.

For this project, the rut depth results are averaged to the 10 m pitch and are converted to an I_3 index according to VSS 40 925b using the curves for RP priority roads. The transversal evenness index I_3 showed that the reference Section and Section B were mostly in good condition along with Sections A, C and D that were found mostly in average condition. The I_3 index was assessed according to the classification (road class HVS) as shown in Annex § 11.4.

Macrotexture

The 3D profiles are used to determine the average PMT texture and PMP profile depths according to VSS 40 511a (ISO 13473-1) per 25 x 25 cm² area. These results are then averaged at a 5 m pitch and by traffic lane: in the middle of the traffic lane, left and right wheel tracks and left and right edges.

Results: Surface Degradation - Index I_1

The results of the various indices are presented in Figure 14 to Figure 17. The results of the surface degradation Index I_1 are shown in Table . The index I_1 results showed for all test sections both reinforced and unreinforced a mix of good and average conditions.



Figure 14: Plan view of surface degradation Index surface degradation, Index I_1

Results: Longitudinal flatness of the road section – Index I_2

The plan, profile and details of longitudinal evenness index I_2 results as shown in Figure 15 revealed for the reference section parts of critical condition along with a mix of good and average conditions. For Section B a slight part in one direction was identified to be in sufficient conditions while the rest of all Section showed a mix of good and average conditions.



Figure 15: Plan view of the longitudinal flatness of the road section, Index I_2

Results: Transverse flatness of the road section – Index I_3

The transversal evenness index I_3 presented in Figure 16, shows that the reference Section and Section B were mostly in good condition along with Sections A, C and D that were found mostly in average condition.

The condition indices I_1 , I_2 and I_3 measured using the ARAN measuring vehicle showed that the test sections both including reinforced and unreinforced are classified between good and average conditions. However, based on the longitudinal evenness index I_2 the reference section has been identified partially in critical condition along with a mix of good and average conditions. A small portion of Section B in one direction was identified to be at a sufficient condition level and the rest of the sections showed a mix of good and average condition. The transversal evenness index I_3 of the reference Section and Section B has shown to be in good condition along with the Section A, C and D that were identified mostly in average condition.

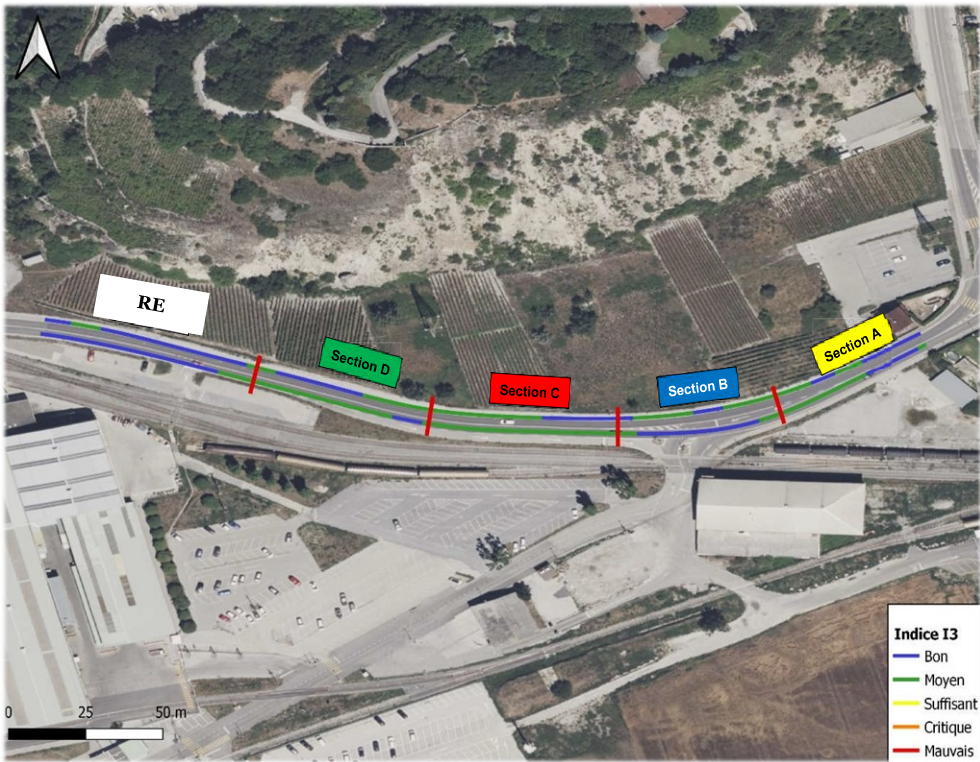


Figure 16: Plan view of transverse flatness Index I_3

Results: Average Texture Depth (PMT)
The details regarding the average texture depth of the test section is presented in Figure 17.

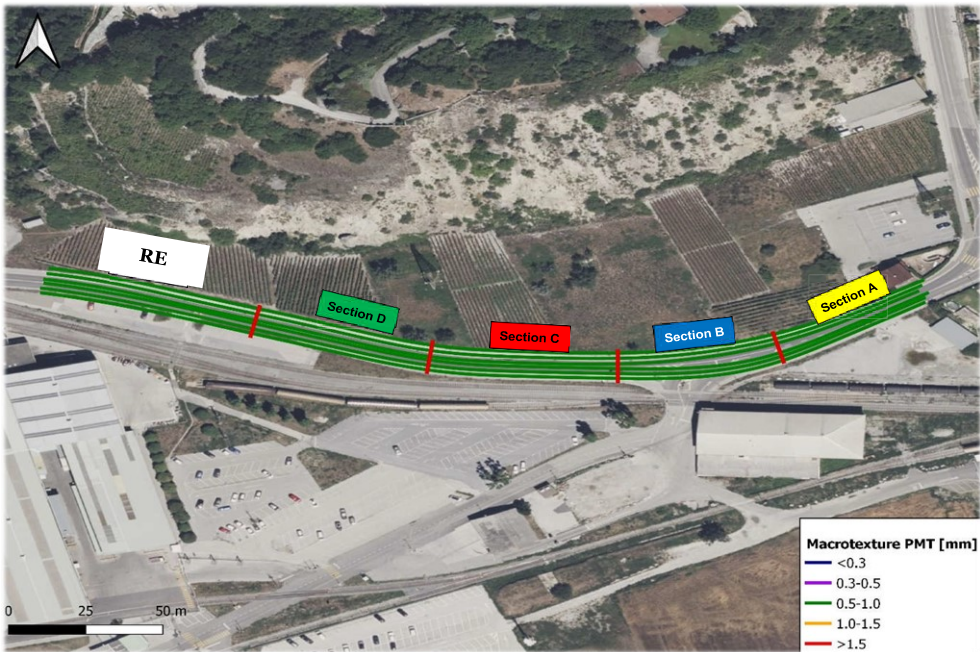


Figure 17: Plan of average texture depth

3.2.3 Falling Weight Deflectometer (FWD)

The pavement was characterized by load bearing by FWD tests. The tests were carried out in both directions of traffic along three axes (left tread, track centre and right tread), with a measurement pitch of 10 m. The falling weight deflectometer FWD measurements was conducted, and the results were back calculated at a reference temperature of 15 °C.

The measurements of falling weight deflectometer FWD revealed that the values in the direction of Susten, were found to be comparatively higher than in the direction of Sion. The deflection values for unreinforced (reference) Section were found to be slightly lower than the reinforced sections showing an improved interlayer well bounded condition. The deflection values from 2021, 2022 and 2023 did not correspond and did not follow a specific trend. The results of the yearly comparison showed that for some of the reinforced sections the values tend to increase while other values decrease compared to the campaign of previous years. As a comparison, a summary of the results in terms of corrected deflection D1 is given in Figure 18.

The back calculated elastic moduli results shown in Figure 19 revealed that overall a slightly higher moduli values of the reinforced grids Section were found in comparison with reinforced grid except Sections A, where the values are slightly above the other reinforcement grid Sections. The back calculated elastic moduli results of asphalt concrete layer of unreinforced section showed slightly higher values compared to the reinforced this might be due to the intervention of reinforcement at the interface. The reinforced sections irrespective of the grid type used revealed slightly higher back calculated elastic moduli values that correspond to the interlayer grid materials at the interface. However, when referred to the corresponding unbound aggregate layer and soil foundation, the reinforcement grid with product A showed slightly higher moduli values.

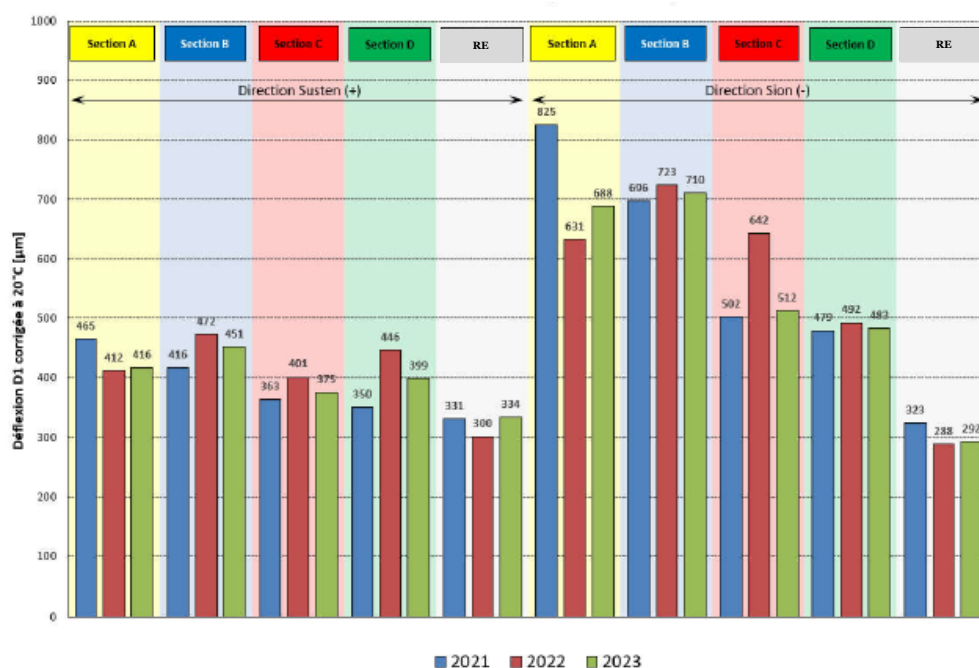


Figure 18: Deflection measurements of year 2021, 2022 and 2023 using FWD

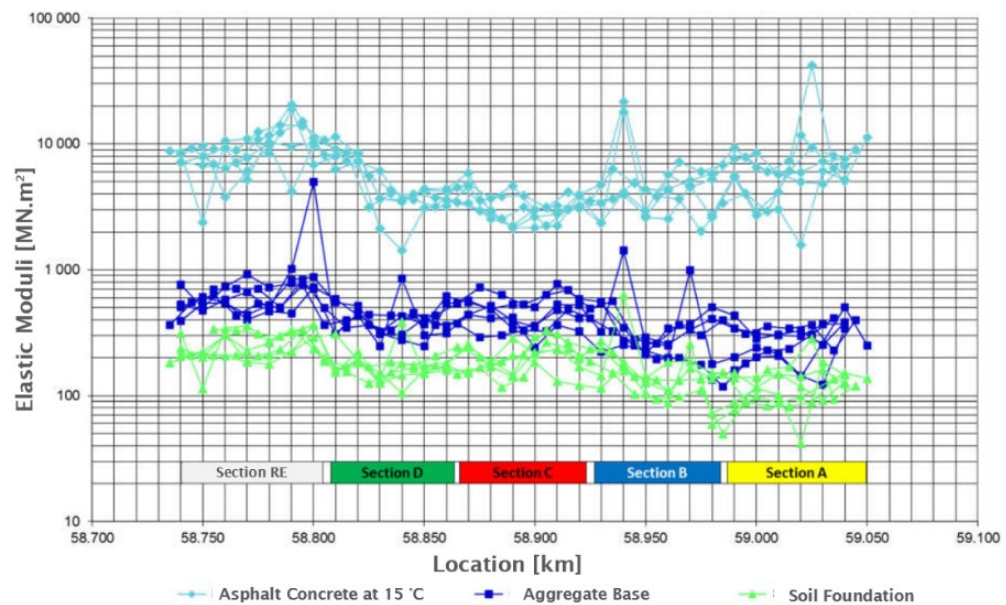


Figure 19: Back calculated elastic moduli of test section

3.2.4 Crack evaluation

After construction of the test track in October 2021, the cracks were evaluated during a visual inspection conducted in April 2023. An ARAN survey was also performed in November 2023, to measure the cracks distribution. The cracks were found to be both in longitudinal and transverse direction as shown in Figure 20. The cracks in the reference section were mostly in transversal directions while, apart from very few transversal cracks in Section B, the cracks in the reinforced sections were mostly in longitudinal direction.

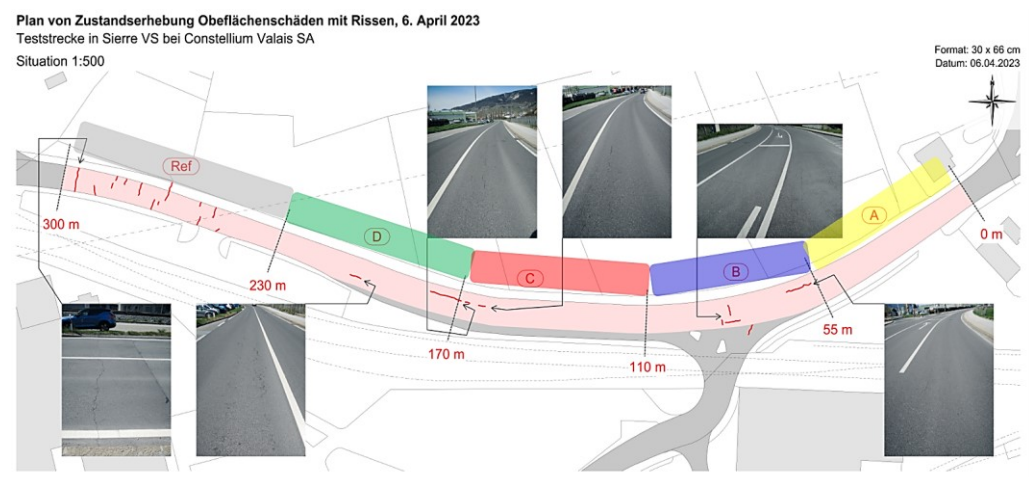


Figure 20: Test Section Visual Condition Survey (06.04.2023)

The analysis of crack appearance as shown in Figure 21 after the construction of the test section is comparable with the available data of ARAN survey conducted in 2019 by the Canton Valais (before the construction of test track). However, the previous data of 2019 was taken in only one direction, which allows the comparison of cracks before

and after construction in only one direction. i.e. direction Sierre. The situation in Section A shows (cf. Figure 22) that before construction i.e. 2019 measurements (blue color lines), there are very few cracks appeared in transverse direction in 2023 (red color lines).



Figure 21: Surface Crack Evaluation

The cracks as shown in Figure 23 in Section B both in transverse and longitudinal direction. The cracks in transverse direction appeared at the same location as detected during the survey in 2019. This confirms the initiation of reflection cracks followed by a crack in transverse direction. The situation as shown in Figure 24 at Section C shows the appearance of small cracks in the middle and a longitudinal crack in the direction Sion which cannot be compared as the previous data is only available in the direction of Sierre.



Figure 22: Crack situation in Section A



Figure 23: Crack situation in Section B

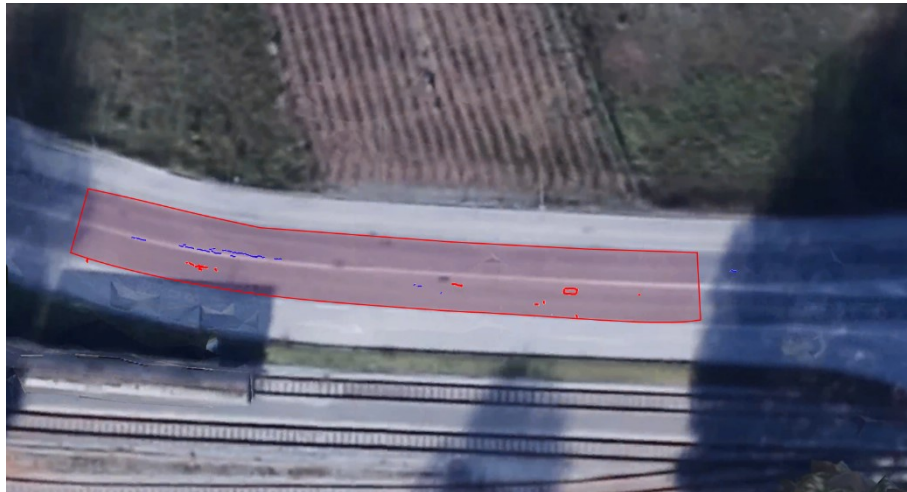


Figure 24: Crack situation in Section C

The cracks in Section D as shown in Figure 25 are distributed in longitudinal direction, however, the cracks before construction in the direction of Sierre were noticed to be both in transversal and longitudinal direction.

The situation in terms of crack appearance Figure 26 in the reference section is worse in comparison to the sections with reinforcement. Most of the cracks that appeared in the reference section are distributed in transversal direction. Many of the cracks in transversal direction are reflected at the same location when compared with the cracks detected before the construction of the test track.



Figure 25: Crack situation in Section D

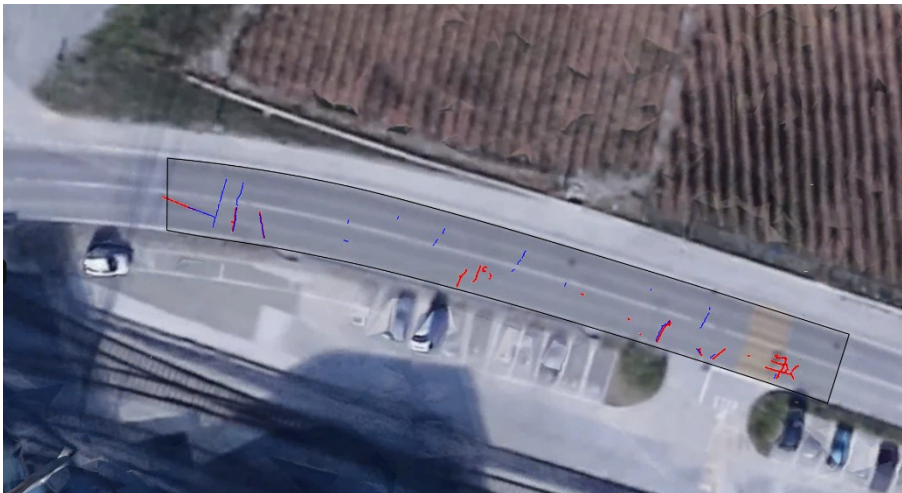


Figure 26: Crack situation in Section Ref

For a more accurate crack evaluation the ARAN data were further processed using imaging technique. In this evaluation the number of cracks was compared and evaluated based on the number of cracks before the construction of the test track. Note that the comparison was made based on the available pre-construction data in 2019 in one direction only (i.e. towards Sierre). The columns in Figure 27 show the number of pixels based on the ARAN measurements in 2019 before and in 2023, two years after the construction of the test track. As can be seen from the figure, the number of cracks in 2019 (based on the pixel count) is much higher in the reference section compared to all reinforced sections. However, the measurements in 2023 show that the percentage of cracks after 2 years is still higher in the reference section in comparison to the reinforced sections.

Figure 28 shows the number of cracks in 2023 as a percentage of the number of cracks in 2019 for all sections. As mentioned earlier the highest percentage with ca. 61 % can be found for the reference section, followed by Section B with a percentage of 50 %. Section A has the lowest number of cracks relative to the number of cracks before construction (ca. 9 %), while the percentage for Section C is ca. 25 %, and 33% for

Section D. The percentage of 9 % for Section A does not include a construction joint that re-appeared on the surface and therefore is not related to grid performance. Since for the performance evaluation of the grids the number of reflective cracks is relevant, the percentage of reflective cracks (comparing the number of cracks before and after construction) for each section was evaluated as shown in Figure 29. Again, the reference section without reinforcement has the highest number of reflective cracks (43 %) followed by Section B with ca. 20 %. In Section A, the number of reflective cracks is ca. 2 % (excluding the construction joint, which is not considered a reflective crack), while in Section C and D no reflective cracks can be observed. The investigations revealed that the installed reinforcements are effective in preventing or at least delaying the appearance of reflective cracks.

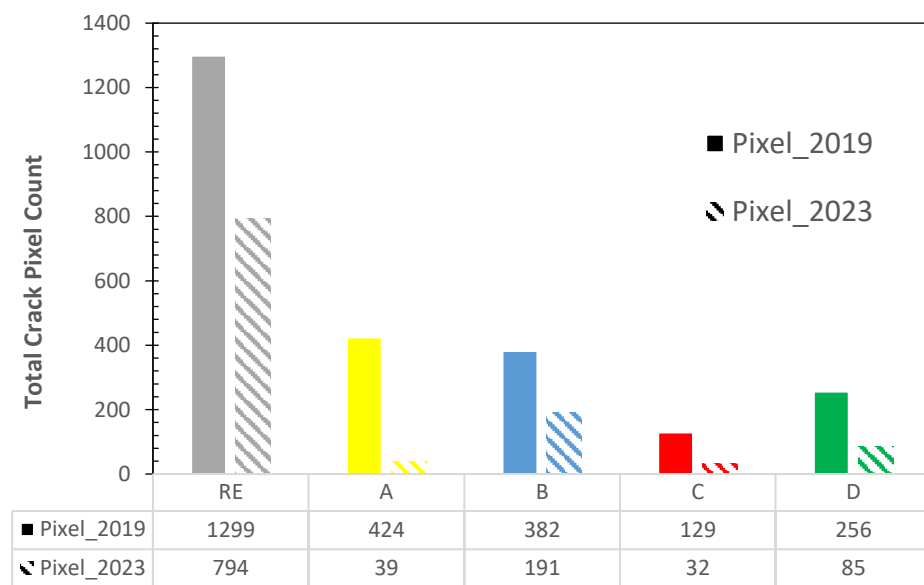


Figure 27: Surface crack pixel value measured using imaging technique

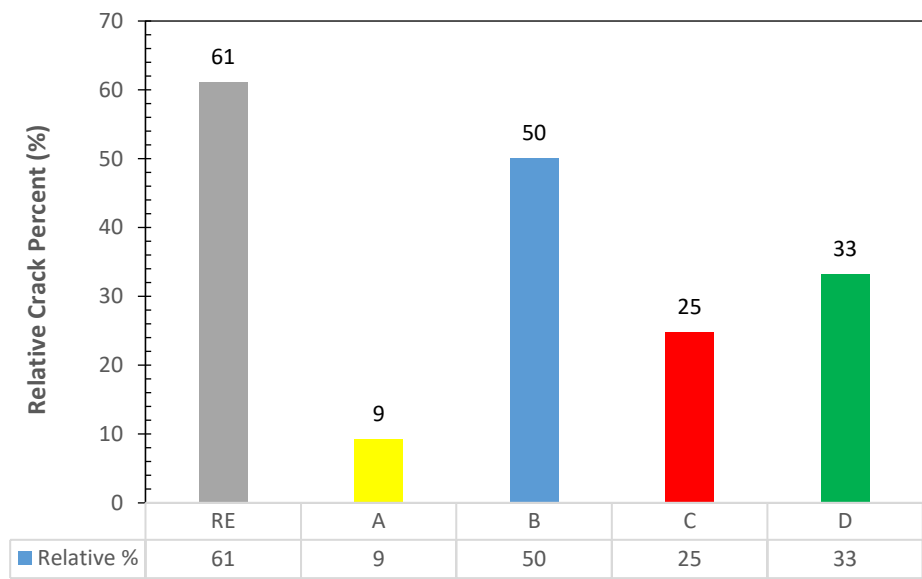


Figure 28: Surface crack pixel value in relative percentage

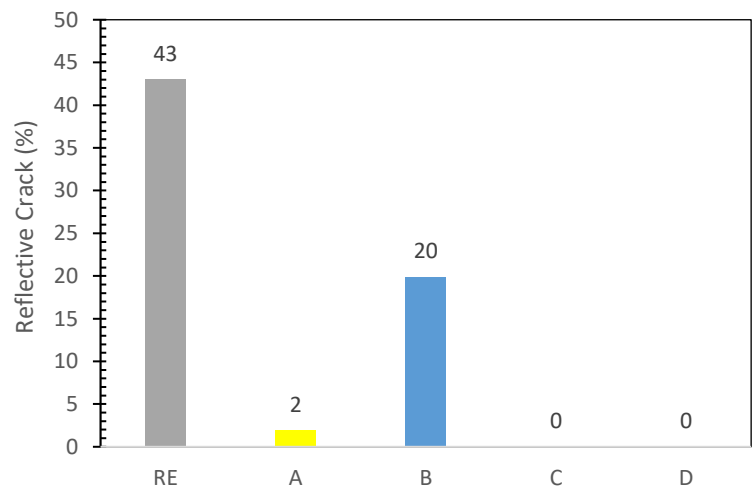


Figure 29: Reflective surface crack pixel value in relative percentage

3.3 Visual inspection in 2024

A visual inspection of the test track was carried out in May 2024. It was found that the test track has visually a high traffic load, particularly considering the number of heavy vehicles.

It was also confirmed that the individual test sections have very different geometries (bends, junctions, etc.) and therefore also significantly different traffic loadings. Examples include the roundabout presence at the beginning of Section A and turn-off lane in Section B must be mentioned.

In some cases, clear surface damage was also visible in the form of raveling and aggregate loss, indicating that the surface course mix had problems. As shown in

Figure 31 the construction joint crack at the beginning of Section A reappeared on the surface.



Figure 30: Cracks at the intersection of turning lane in Section B



Figure 31: Construction joint crack at Section A

According to the findings of the inspection in May 2024 it is difficult to compare the reinforcement grids installed in the different sections with each other, because of the very different geometries in the form of curves and junctions and thus different traffic loads for the individual sections. In this context, the roundabout presence at the beginning of Section A and turn-off lane in Section B must be mentioned.

3.4 Summary

After two years of monitoring, the following observations can be made:

- On the basis of only one core per product, the interlayer bond test results showed that the reference section has the highest value in comparison to the reinforced sections irrespective of the reinforcement type used and therefore the best interlayer bond properties. However, the product used in Section A has the higher values followed

by Section B, while Sections C and D never seem to be able to reach the standard requirement of 15 kN.

- From the results it is also clear that the interlayer bond properties improved over time, a fact that is in accordance with earlier investigations (Raab et al., 2017). The increase in terms of shear force for systems A, B and the reference are in the order of 40 %, while the increase for System C and D is 66 % and 76 % respectively.
- The condition index I_1 changed significantly from "good" to "average" over a large part of the test track more specifically in Section C and D, which can partly be contributed to the appearance of various cracks in all sections.
- For Index I_2 no significant changes are noticeable.
- Index I_3 moved from "good" to "average" for a large part of the test track.
- The PMT values (average texture values) remain relatively homogeneous over the entire test track.
- At the structural level, apart from an isolated point, the service life is more than 20 years. The level of deflection remains similar from one year to the next, regardless of the sections.
- The results of crack analysis show that there are cracks that are reflected at the same location where cracks were detected before construction of test track.
- The cracks in the reinforced Sections were mostly distributed in longitudinal direction. However, the cracks in the reference section were mostly in transversal direction.
- It was observed that the crack intensity was higher in the reference section in comparison to the reinforced sections. This shows the positive influence of the reinforcement against crack propagation.

3.4.1 Field Tests

In-situ test sections can be constructed at low cost. They are subjected to a realistic stress, so that a direct comparison of the stress effects between sections with and without inlay is possible. For concrete statements on the development of damage, however, a very long monitoring period is always required, as only an investigation in real time is possible. The use of traffic load simulators can accelerate this process to a certain extent.

Chazallon et al. (2019) tested different variants of asphalt reinforcement using five test fields. For this purpose, samples were taken from the test fields and laboratory tests were carried out on these samples.



Figure 32: Installation of the test fields with asphalt reinforcement (Chazallon et al. 2019)

The results showed a great dependence between the grid and the bituminous mix in terms of bonding and observed damage. The choice of the type of grid should be based on these two properties. To be able to make statements on the estimation of the service life of pavements with asphalt reinforcement, additional tests were carried out with a large-scale testing facility. The six loaded test fields (with or without asphalt reinforcement) were equipped or instrumented with sensors so that monitoring is possible during the test. Furthermore, the possibilities for deconstruction and recycling of the materials were analysed to better quantify the environmental balance of asphalt reinforcements.

Within the scope of test fields of asphalt pavements with and without asphalt inlays, the crack-delaying or crack-preventing effect of asphalt inlays could be demonstrated (Mollenhauer et al. 2023). Reflection cracking on the asphalt surface layer occurred significantly earlier and to a greater extent in the reference variant without asphalt inlay than in the variant with asphalt inlay. The crack-preventing or -decreasing effect of the different asphalt layers could be clearly seen in the pavement samples from the large-scale test pavements.



Figure 33: Test Field during core sampling

Within interlaboratory tests of six participating institutions to characterize the mechanical properties of grid-reinforced asphalt pavements the importance of considering the debonding and the reinforcing effect of the grids were highlighted (Canestrari et al. 2018). The debonding effects were evaluated by measuring the interlayer shear strength with five different devices, at different test temperatures and

speeds. The results showed that the reduction of interlayer shear strength was statistically significant only for the glass fiber reinforced polymer grid, which is characterized by higher thickness and torsional stiffness with respect to the carbon fiber/glass fiber grid, whose mesh is extremely flexible and deformable.

The reinforcing effects were evaluated using both cyclic and monotonic bending tests on double-layered beams characterized by different dimensions, at different test temperatures and speeds. These results showed that the permanent deformation resistance and the flexural toughness of the FP-reinforced system were significantly higher with respect to that of both unreinforced and CF-reinforced systems.

3.4.2 Test Sections

On an international level, a large amount of information is available from projects in which asphalt sections with and without asphalt reinforcement were produced as part of a construction project. However, these projects were usually not scientifically monitored over a longer period, so that the findings are only indicative. In the following, an overview is given of studies in which a detailed statement was made on the effect of asphalt reinforcement in in-situ investigations. This is mainly based on a visual condition analysis of the surface and investigations of the stiffness of the superstructure, e.g. using the falling weight deflectometer (FWD) method (Tazl 2013). Nunn & Potter (1993) investigated the effect of Stress Absorbing Membrane Interlayer (SAMI) construction, reinforcement products, surface course thicknesses and polymer modified bitumen products on various motorway and rural road sections in the UK. Visual documentation methods and FWD measurements are used, with a maximum study period of three years. The investigations show that crack development starts from the surface and continues through the asphalt surface. In the case of a superstructure with 80 mm asphalt, there were no signs of cracking in the grid-reinforced section after three years, while cracking was already evident in the unreinforced reference section. Overall, polymer-modified bitumen products and greater overlay thicknesses lead to significantly reduced cracking. Cracking is particularly pronounced during the cold winter months, when the pavement is brittle and more susceptible to stresses (Nunn & Potter 1993).

In another study (Laurent & Serfass 1993), the effect of asphalt interlayers to reduce cracking on a motorway section in France was investigated. Compared to the reference section, the fleece inlays show a reduction and equalisation of reflective cracking. The authors recommend a high injection rate for the inlay to achieve a stress-relieving effect and an adequate bond with the surrounding asphalt layers.

In Austria, on several investigation sections with a length of 300 to 400 m, the conventional superstructure technique (120 mm) was compared with grid-reinforced (Polyfelt PGM 14, Tensar AR-G and steel grid "Mesh-Track") thin-layer superstructures and SAMI construction methods (60 mm) over a period of three years. The plastic asphalt grid showed no signs of reflective cracking after two years, while the installed steel grid led to significant cracking in the pavement. Initially, the 120 mm thick superstructure showed the best results, but after about 2.5 years, the values became similar to those with a fleece liner and a layer thickness of 60 mm (Herbst, Kirchknopf et al. 1993).

O'Farrell (1996) investigated eight different construction methods on a motorway in Great Britain, four of which were carried out with asphalt grids or fleeces in a layer bond. According to his analysis, asphalt interlayers between the surface and binder

courses have a lower crack resisting effect than corresponding interlayers between the binder and base courses. Furthermore, the resistance to re-cracking is increased by laying the asphalt inlays over the surface compared to localised laying. O'Farrell cautions that no meaningful method is yet available to document and analyse resistance to reflective cracking, so comparison of different construction methods remains problematic.

In a Belgian study (Vanelstraete & Decone 1996), twelve different road construction projects with asphalt overlays were analysed. In this study, the superstructures of asphalt pavements and continuous concrete layers with asphalt inlays showed no or very little reflection cracking, while the concrete slabs led to significant reflection cracking on the pavement surface. The authors state that the thickness of the overlay is one of the most important factors in reflection cracking, with a 15 mm thick overlay leading to significant cracking, while a 120 mm thick overlay showed no signs of cracking.

Raab & Partl (2004) focused in their studies on the interlayer properties between concrete pavements rehabilitated with asphalt and an intermediate layer consisting of steel mesh, a glasfibre geotextile or a bituminous treatment. Therefore, a concrete motorway in Switzerland got rehabbed by adding an asphalt surface layer. This section was divided into six test sections including different types of milling, reinforcement grids, and tack coating. Afterwards, drilling cores were taken, and the adhesion properties were tested with the pull-off test. The results showed the importance of the adhesion performance of rehabilitated concrete roads. Furthermore, the authors indicate the importance of choosing systems and construction techniques which minimize the negative effects on adhesion.

In 2011, the Hessian Road authority Hessen Mobil planned to carry out a rehabilitation using reinforced asphalt layers on the rural road L3079 in Hesse. The existing asphalt layers were milled off by approx. 1-2 cm and rehabilitated with a binder course AC 16 B S and a surface course AC 8 D S. A pre-bituminised asphalt reinforcement was used on the new bearing course. To verify previously conducted performance tests as well as the dimensioning approach for the investigation of asphalt inlay systems, Mollenhauer et al. (2023) examined removal samples from existing pavements that had been under traffic for many years. Already after a service life of 9 years after the renewal of the asphalt surface course and asphalt binder course with asphalt inlay, renewed cracking could be detected. However, it was possible to prove on pavement samples that these cracks had not penetrated from the base, but had newly developed in the renewed, cavity-rich binder course. The asphalt layer, made of a combination product with glass grid, apparently interrupts both old cracks originating from the base and the downward growth of new cracks in the asphalt binder course in all pavement samples.

3.5 Research needs

Although the effect of asphalt layers has been proven in practice in many cases, there is currently no verified theory to describe the relationships and mechanisms of action. In connection with this, there is a need for meaningful investigation and testing methods that enable a direct comparison of different asphalt layers (FGSV 2013).

The research presented here, which will serve as the basis for the study proposed here, provided some very interesting preliminary data, but several other important questions could not be answered conclusively:

- As adhesion is very important, the way the grid is applied has a great influence on the quality of the bond and the effectiveness of the reinforcement. The effect of adhesion between the grid and asphalt in a pavement structure needs to be quantified.
- The quantitative estimation of the contribution of the reinforcement to the bearing capacity (fatigue strength, crack resistance, deformation resistance) of the pavement is missing.
- The work carried out does not provide a methodology to consider the reinforcement grid in the design of reinforced pavements.
- The weight traffic simulator MMLS3 is a relevant test, but still the result needs to be statistically supported by repeated tests.
- The tests performed in the field bring interesting elements but are somewhat far from what happens in a pavement in practice (artificially pre-cracked pavement to highlight the effects of reinforcement more quickly, prompts that are not representative of real traffic on site, especially due to load channels and loading frequency). In situ test rigs are therefore necessary to be able to provide the missing information for updating the standardisation in the field.

3.5.1 Consideration of reinforcement grids in the design process

Despite many years of experience with grids for asphalt reinforcement, relevant methods for the evaluation and design of pavements with asphalt reinforcement are lacking. This is a major obstacle to the acceptance and integration of these systems in road construction (5). Furthermore, the inventory and comparison of the different methods and practices used internationally illustrate that there is no general calculation method for the different types of asphalt reinforcement to predict the long-term behavior of grid-reinforced pavements.

The present research project will therefore rely on the literature and the few existing case studies. Nevertheless, a small number of calculation models already exist:

- The Danish Road Design Manual addresses the problem with an empirical approach based on triaxial laboratory tests (FOLKBRO 2017).
- The reflective crack propagation calculation software (ARCDISO) is able to analyze maintenance solutions against reflective cracking (www.arcdeso.nl).
- The REFLEX software uses an equivalent layer for reinforcement to calculate an asphalt/reinforcement steel composite system.
- The University of Nottingham has developed an approach to consider the propagation of a crack considering the three possible crack shapes (Kakouris & Triantafyllou 2018).

The methods listed above contain some principles that can form a solid basis for the current research project. However, these methods are not directly applicable for the purposes of this research project, so a detailed analysis of the transferability of the results is required.

3.5.2 Effects of reinforcing grids on service life

The presence of a reinforcing grid in the interface of asphalt layers reduces/modifies their adhesion by changing the structural behavior of the pavement. Laboratory tests can be used to investigate the shear strength between asphalt layers to quantify the quality of the system. However, large-scale observations showed significant discrepancies with laboratory observations, with the latter systematically providing values approximately 40% higher than large-scale observations (Raab et al. 2017). This may be due to damage to the grid during installation and/or repeated traffic-related stresses.

The Mobile Load Simulator Model (MMLS3) used in the ASTRA 2011/011/011 (Raab et al. 2017) study to test pre-cracked asphalt pavements (reinforced and unreinforced) proved to be a representative tool to highlight the contribution of reinforcement to increase durability against fatigue and to allow discrimination between the different reinforcement products tested. However, the result needs to be statistically supported and confirmed on a variety of reinforcement meshes and dam structures. In addition, further information can be obtained from this test. For example, instrumentation adapted to the specimen to capture the moment of crack formation and measure the speed of its propagation would allow us to apply behavioral rules from fracture mechanics. This would allow us to get back to basic quantities of cracking behavior of asphalt mixtures, which are necessary for the design procedures.

Finally, there is a need to observe representative data from large-scale tests to correlate laboratory observations with the behavior of real reinforced pavements. There is also a need to investigate the practical aspects of installation, which should allow reinforcement meshes to be better considered in the design of reinforced structures. Instrumentation and its monitoring over time can be used to collect the data necessary for modelling.

3.6 Summary

In this chapter, the basics of the use of asphalt reinforcements in road construction were presented. For this purpose, the different construction methods, and materials as well as their functioning of reinforcement grids were first explained. Subsequently, the state of knowledge from a national and international perspective regarding the use of asphalt reinforcement was compiled.

In Switzerland, two major research projects were carried out to investigate the use of asphalt reinforcement in the maintenance management of base and surface courses. The main findings from the investigations on test structures in the laboratory under simulated traffic loading and on test fields in situ under accelerated traffic loading showed that asphalt reinforcements can delay the occurrence of cracks and thus extend the service life of asphalt pavements. In both laboratory and field tests, asphalt reinforcements without SAMI were found to be more effective than those with SAMI. Laboratory and field tests showed that reinforcement inserts represent a separation layer that can lead to impairment of the layer bond. However, based on the field tests, it could be concluded that with careful workmanship, layer bond values can be achieved that meet the standard requirements.

Internationally, the use of asphalt layers in asphalt road construction has been investigated since about the beginning of the 1990s. Nevertheless, it has to be stated

that only a few independent studies on the use of asphalt reinforcement can be found so far. Most of the available sources are provided by the manufacturers. The research results show that the use of reinforced asphalt layers lead to a delay of crack propagation. However, the studies also identified reduced interlayer bonding due to nonwovens, composites, and grids. Furthermore, there are hardly any studies on the long-term behavior of superstructures with asphalt reinforcement, as the observation period within the framework of research projects is usually limited to a few years. An alternative to this is load tests with which time-elapsing load cycles can be applied and their effect on the material behavior can be investigated. From the results of the literature research, the need for research regarding the use of asphalt reinforcement could finally be derived.

4 Methodology

The different stages involved in the project investigation are presented in Figure 34. After an extensive update of knowledge and the study of recent representative literature (Task 1), an experimental investigation in the laboratory was conducted (Task 2). The quality of grid/asphalt composites (adhesion between layers), the investigation in terms of fatigue strength and cracking was conducted in a laboratory. The full-scale test Sections (Task 3) was focused on the practical aspects of installation of the grids and then monitoring over time for deterioration due to mechanical and climatic stresses. Samples taken from test sections was also analyzed in the laboratory, with the aim of highlighting the deterioration of the composite (material fatigue, deterioration of the adhesion between the layers, damage to the reinforcement grid due to installation and exposure to traffic). The various data collected in the laboratory and in situ was used to refine and update interlayer reinforced pavement modelling (Task 4).

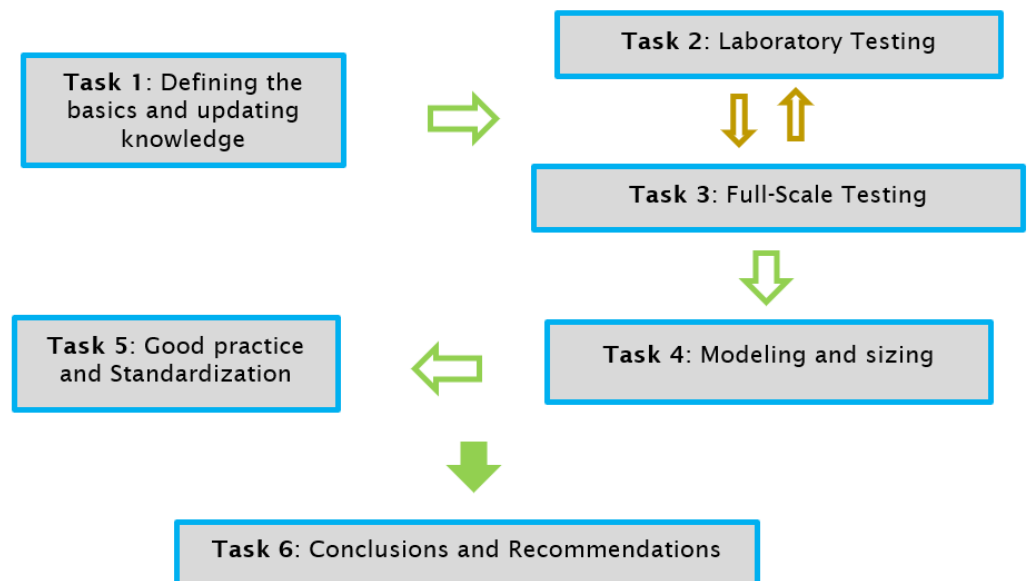


Figure 34: Different stages of the project

The experimental plan in Figure 35 shows the laboratory and field investigation assigned to each research partner.

In Task 5, the accumulated data and lessons learned were ultimately used to develop a detailed methodology and recommendations for the design and implementation of maintenance/rehabilitation activities incorporating reinforcement grids while considering the in-situ conditions (climatic conditions, condition of the receiving layer, installation conditions, etc.). Finally, substantial recommendations to update the standardization on the design of pavement structures (VSS 40 324) and on the maintenance/reinforcement of pavements (VSS 40 733) have been proposed.

The details regarding laboratory tests that were performed on a different size specimen at cm scale can be found in Figure 35. Conventional test for measurement included asphalt mix stiffness and interlayer bonding. The laboratory and field cores were evaluated for the quality control and layer adhesion properties. Laboratory test at slab size (m scale) mmls3 slab test for grid cracking evaluation and 4-point bending beam for reinforced grid cracking evaluation with and without overlap length. Field test with distress measuring campaigns survey included ARAN, FWD measurements and core extractions were carried out. Modelling of multilayered grid slabs behaviour under load was simulated. The LCA analysis was finally performed to incorporate conclusions & recommendations including best practices for pavement maintenance and rehabilitation activities.

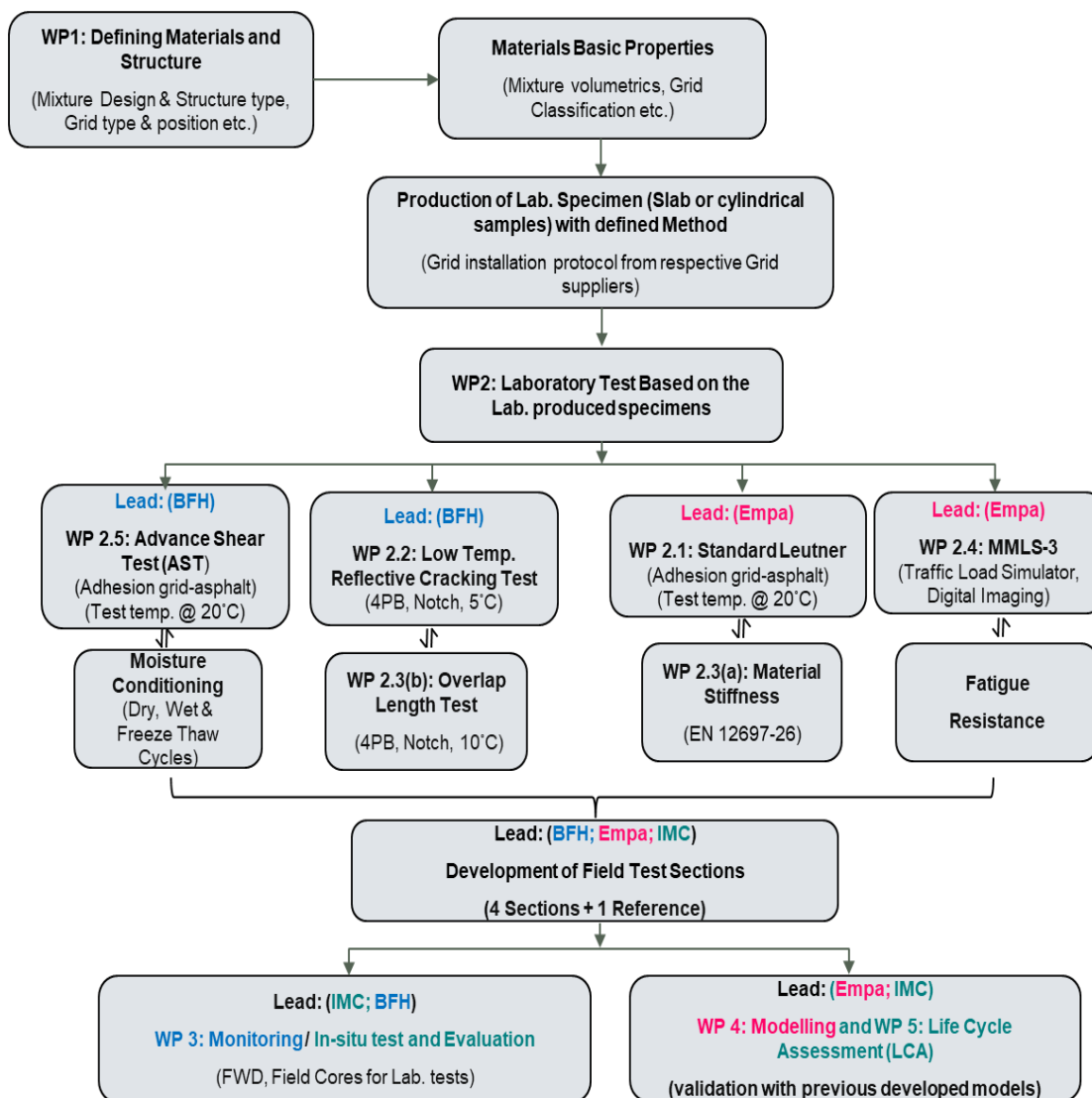


Figure 35: Methodology flow chart

5 Materials

5.1 Asphalt concrete

For each of the tests carried out in this project, the same asphalt concrete mixtures AC 8 S and AC B 11 S, were used. The mixtures used were obtained from an asphalt plant in Hinwil. To that end, 1.5 ton of each material was picked up from two batch using carton boxes (see Figure 36). Then, different properties of the bitumen and the mixtures were established in the laboratory.



Figure 36: Collection of the asphalt mixtures AC 8 S and AC 11 S from an asphalt producing plant in Hinwil

On one hand, the bitumen was extracted from both mixtures and its content and properties like penetration and ring and ball were determined. The maximum density, Marshall-Stability values, granulometry of the mixtures were also obtained. The results, together with the declaration sheet from the asphalt plant can be found in the Annex § 11.1.

On the other hand, a property like the stiffness complex modulus which is important for the accurate modelling of a pavement under load were also obtained and the results are available in Annex § 11.1.4. An explanation about how these tests can be used to simulate temperature dependent elasticity and viscoelasticity in the models developed in this work are available in § 7.1.4.1.

5.2 Reinforcement grids

Four different types of reinforcement grids were used. The summary of the most relevant mechanical properties is shown in Table 2. In order to ensure the anonymity of the results, the grid products are labelled with the letters A, B, C and D.

Reinforcement grids							
Product	A		B		C		D
Tack coat- Field (residual) [g/m ²]	180		690		none*		420
Tack coat- Lab (residual) [g/m ²]	180		480		none		250
Strand direction	L	T	L	T	L	T	L T
Strand Material	Glass	Carbon	PET		Glass		Glass Carbon
Mesh size [mm]	15	15	40		10		18 18
Tension strength [kN/m]	120	200	50		100		120 200
Elongation [%]	3	1.5	12		3		3 1.5

*Emulsions was only applied on the edges and top of the grid (600 g/m²)

Table 4: Mechanical properties of the reinforcement grid used in the project

5.3 Tack coat

To prepare the specimens in the laboratory, for the case where no reinforcement grid was applied, the tack coat was a polymer modified cationic emulsion with 60 % of residual bitumen. System C includes a self-adhesive membrane and therefore doesn't need tack coat. For the reference specimens without the application of any reinforcement grid, the emulsion was applied in a concentration of 200 gr/m² residual bitumen. The tack coat at the interface for prepared specimens with reinforcement grids was based on the recommendations of suppliers depending on the type of different grid products used. The details regarding type and amount of tack coat applied to specimens prepared with different products including reference (without grid) are given in Annex § 11.5

Reinforcement grids installation details

Product	Location	Coating Material	Dosage of Coating Material (Residual)	Chipping
A	Field Track	Cationic emulsion with a residual bitumen content of 60 %	180 g/m2	No
	Laboratory	Cationic emulsion with a residual bitumen content of 60 %	180 g/m2	No
B	Field Track	Cationic emulsion with a residual bitumen content of 60 %	600 g/m2	No
	Laboratory	Cationic emulsion with a residual bitumen content of 60 %	560 g/m2	No
C	Field Track	Smart Tape	-	No
	Laboratory	Cationic emulsion with a residual bitumen content of 60 %	60 g/m2	No
D	Field Track	Cationic emulsion with a residual bitumen content of 60 %	420 g/m2	Yes
	Laboratory	Cationic emulsion with a residual bitumen content of 60 %	250 g/m2	No
RE	Field Track	Cationic emulsion with a residual bitumen content of 60 %	350 g/m2	No
	Laboratory	Cationic emulsion with a residual bitumen content of 60 %	200 g/m2	No

Table 3: Grid product installation details

6 Laboratory tests

6.1 Evaluation of the interlayer bonding

6.1.1 Leutner test

6.1.1.1 Specimen preparation

Two-layer test specimens consisting of an AC 8 S surface course and a base course AC B 11 S were produced. The large device rutting compactor was used to compact the asphalt layers, using a double-width compaction frame (width: 395 mm). The mix was heated in the oven and poured hot into the compaction frame. Compaction was carried out in layers using a steel roller designed for this purpose, which was pulled over the rubber tires of the machine (see Figure 37). The asphalt reinforcement was applied between the surface and the base course. An unreinforced slab structure using a tack coat was produced as a reference.



Figure 37: Modified compactor of the large device rutting tester

6.1.1.2 Reinforcement application

The application of the different reinforcement systems was carried out by the manufacturers themselves. The same procedure applies for the specimens produced for MMLS3 testing.

System A: Glass and carbon fiber grid:

The grid was flamed to the underlying asphalt layer. Before laying the grid, 300 g/m² of cationic emulsion had been applied with a brush (see Figure 38).



Figure 38: Application of System A

System B: Polyester grid

The polyester fiber grid was applied on top of the asphalt layer using a dough roller. Before laying the grid, 400 g/m² of cationic emulsion had been applied (see Figure 39).

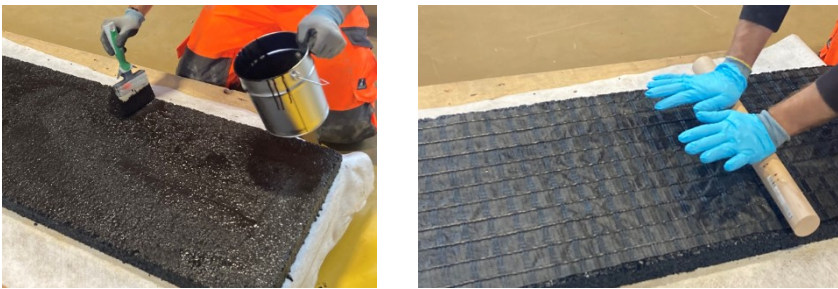


Figure 39: Application of System B

System C: Glass fiber grid with a highly self-adhesive bitumen membrane

Since the highly self-adhesive bitumen membrane acts as tack coat, the grid was directly installed using Empa's laboratory steel roller compactor (see Figure 40).



Figure 40: Application of System C

System D: Glass fiber and carbon fiber grid:

The grid was applied using a roller brush on top of the asphalt layer. Before laying the grid, 250 g/m² of cationic emulsion had been applied. Prior to the application, the emulsion had been heated to a temperature of 60 °C (see Figure 41).



Figure 41: Application of System D

6.1.1.3 Core extraction and testing

From the prepared slabs 6 cores as shown in Figure 42 b) with a diameter of 150mm are extracted of each reinforcement system including the reference. For interlayer bond testing, the direct shear test according to Leutner [lit] and standard EN SN 12697-48 was used with the shear test device modified by EMPA. The modified EMPA testing device can be integrated into a servo-hydraulic testing machine (e.g. a conventional Marshall press) and allows the testing of cores with a diameter between $d = 148$ mm and 150 mm, whereby the two U-shaped shear-jaw inserts can be adapted to the core diameter at millimeter intervals (see Figure 42). During the test, one end of the drill core lies on a u-shaped bearing surface up to the selected shear plane and is pressed against by a pneumatic clamping device with defined pressure, while the other end of the test specimen is not held. The shear force is introduced into the shear plane by the piston via the shear jaw insert attached to a yoke at a speed of 50.8 mm/min. In the process, the test specimen is loaded until it breaks.

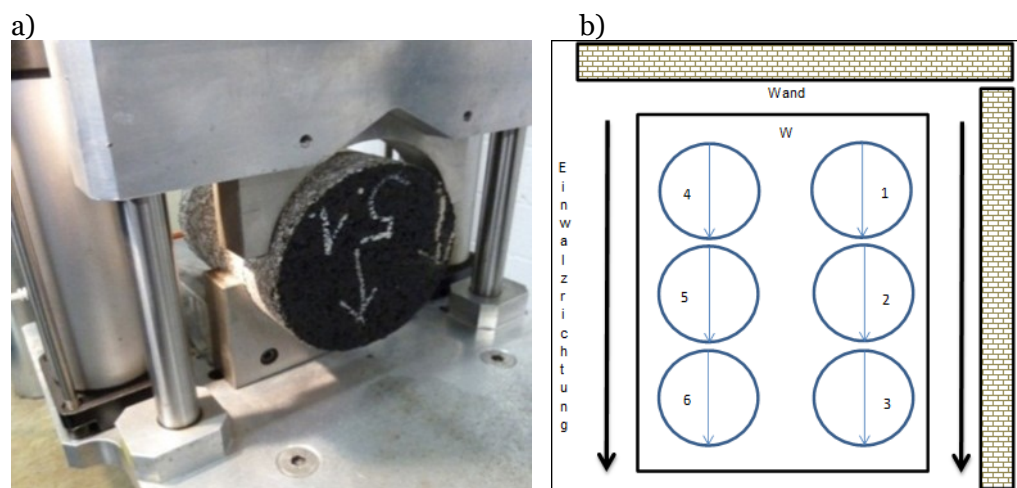


Figure 42: Testing of the layered composite according to Leutner, a) test device, b), coring diagram

6.1.1.4 Results

Figure 43 shows the interlayer bond test results. In the laboratory investigation, the maximum shear forces of the investigated reinforcement systems differed quite significantly. Especially, System A achieved with an average of 33.2 kN very high values, followed by B with an average value of nearly 25.8 kN, while System C performed very poorly with an average max. shear force of 5.8 kN. System D reached an average of 16.0 kN, but regarding the standard deviation, the value of 15.0 kN required by the Swiss standard (SN-EN-12697-48) could not be achieved for all tested samples. System A even outperformed the unreinforced system (reference) which achieved an average max. shear force of 30.3 kN.

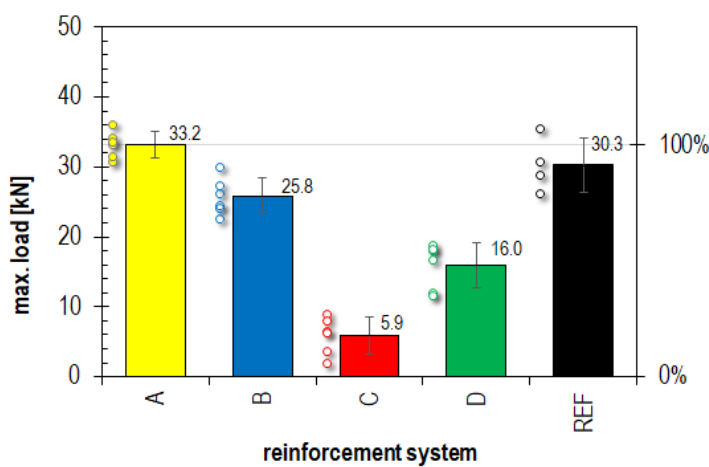


Figure 43: Leutner test results for each reinforcement system

6.1.2 Advanced shear test

6.1.2.1 Description

The test setup of advance shear tester (AST) as shown in Figure 44 was used to assess the adhesion between layers, this by applying a test method that is believed to be more realistic and reliable than Leutner test traditionally used as a standard test in Switzerland and many other European countries. In this project, the AST was used to identify the effects of moisture damage (wet condition) on the bond strength at a reinforced interface with various types of reinforcement grids. The reinforced interface compared with corresponding unreinforced interface was evaluated. It is inferred that the AST device can better simulate the actual loading condition in the laboratory and help to better understand the differences in structural integrity of the System At the interface level while subjected to different environmental conditions.

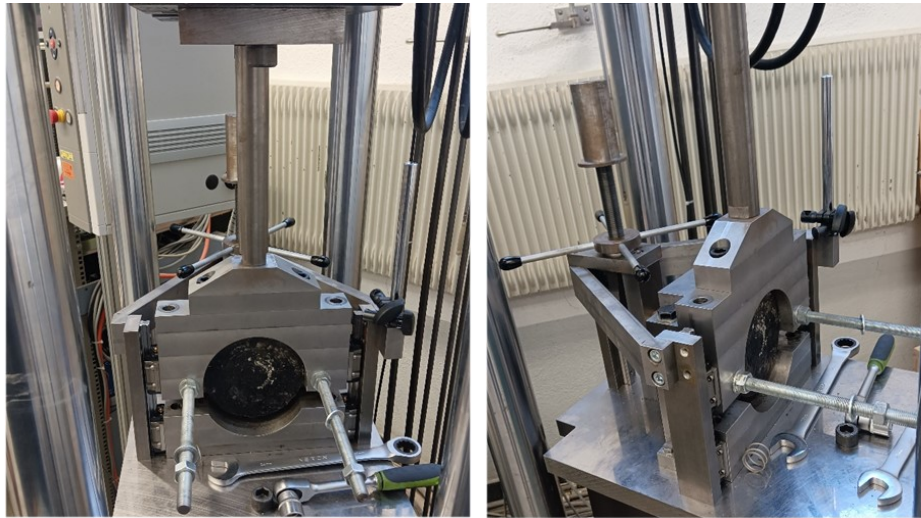


Figure 44: Advance Shear Test Setup

The main feature of this device, which makes it distinguished compared to similar devices (e.g. Leutner), is to resemble the changes in normal stress due to traction and compression along the pavement interface while constrained by the surrounding materials as shown in Figure 45.

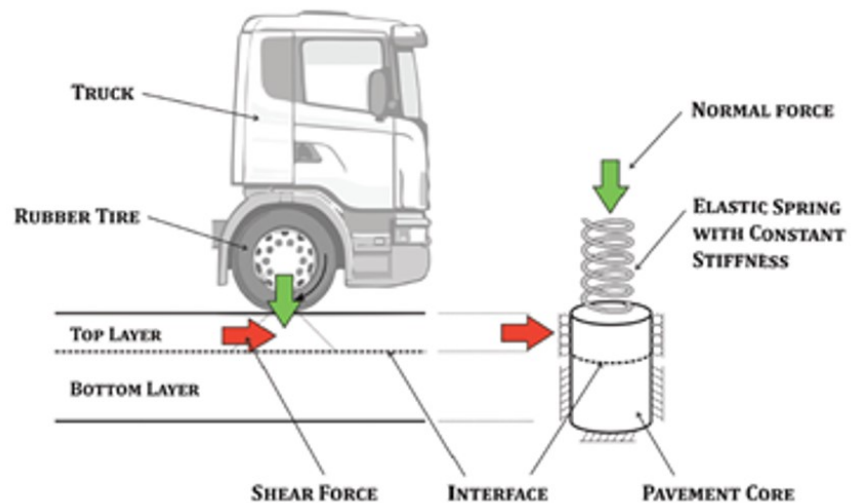


Figure 45: Normal Stress Variability at the Interface under Heavy Loads (Zofka et al., 2015)

The AST device is composed of a stationary and a movable part in vertical direction, both are fixed at the bottom with a base plate, already mounted on the hydraulic press. The distance between these parts provides an adjustable free space in three different gap sizes based on the required thickness of the interface being tested. For the sake of this project, in order to evaluate the bond strength between the bottom layer and the reinforcement grid, the thickness of the interface zone was selected 5 mm, in total. The gap thickness includes the reinforcement grid, the tack coat and a thin part of the bottom layer in contact with the reinforcement grid. The normal force was applied, perpendicular to the interface (shearing plane), through tightening the nuts on the threaded rods, already installed to the movable part, as long as a desirable confinement

pressure was obtained. In addition, the shear force was transferred to the interface in a constant displacement rate of 50 mm/min via a hydraulic press by pulling up the movable part of the AST device up to 15 mm. The shear and normal forces were measured using the load cells connected to the upper loading jaw and outer plate, respectively. Furthermore, the resultant shear displacement in vertical direction and horizontal displacement were measured by means of two external linear variable displacement transducer (LVDT) sensors.

6.1.2.2 Specimen preparation

The installation of grid reinforcement over the slab specimens prepared for advance shear test and crack propagation test were placed according to the laboratory demonstration and the agreed documents prepared with the help of each supplier. The details are provided in the Annex § 11.2.

The double layer asphalt slab specimens were composed of two different hot mix asphalts in terms of maximum aggregate size complying with the SN 640 431-1-NA (EN 13108-1) that were obtained from plant. The upper layer of the slab is composed with a nominal maximum aggregate size (NMAS) of 8 mm (AC 8 S) (gradation curve shown in Annex § 11.1.3) and is placed over a binder course with an NMAS of 11 mm (AC B 11 S). The mixture Marshall stability performance details of AC 8 S and AC B 11S are given in Annex § 11.1.4 and Annex § 11.1.5, Table A1 and Table A, respectively.

6.1.2.3 Reinforcement application

For each of the reinforced specimens, the corresponding reinforced structure including a layer of reinforcement grid at the interface of mixtures were prepared, as shown in **Figure 46**.

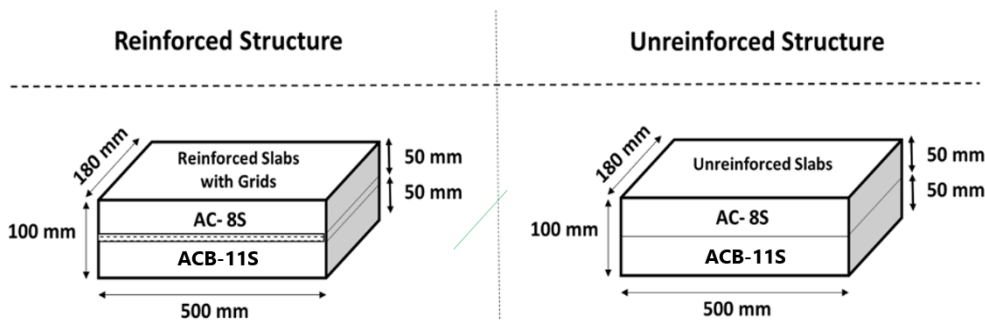


Figure 46: Schematic of the double layer slab specimens

The construction of each slab structure was started by the production of the bituminous bottom layer at a mixing temperature of 165 °C, in a mold of size 500 × 180 × 100 mm. The mixture was compacted by means of the French roller compacting device at compaction temperature of 150 ± 2 °C complying with the type of bitumen (i.e. 50/70). After 24 hours, procedures, defined according to demonstration recommended by each grid supplier, were pursued to install the different type of reinforcement grids on the top surface of the prepared bottom slabs.

The reference slab structures designated as unreinforced, a layer of a cationic emulsion (composed of 60 % bitumen and 40 % water) and as much as 200 gr/m² of the residual bitumen was applied on the surface with the help of a syringe and uniformly distributed

via the edge of a spatula. After waiting 3 hours for complete breakage of the emulsion, the reinforcement grids were placed at the interface of the double layer slab specimen.

6.1.2.4 Core extraction and testing

The slab production procedure was initiated by compacting the upper layer with hot mixture at 150 ± 2 °C over the unreinforced (reference slab) and reinforced grid interface having dimension of $500 \times 180 \times 50$ mm. The double layer slab specimens prepared as described above, three specimens from the central part, with the diameter of 150 mm were cored and processed by cutting and grinding the end faces. The final dimension of specimens derived from each slab structure are schematically shown in Figure 47.

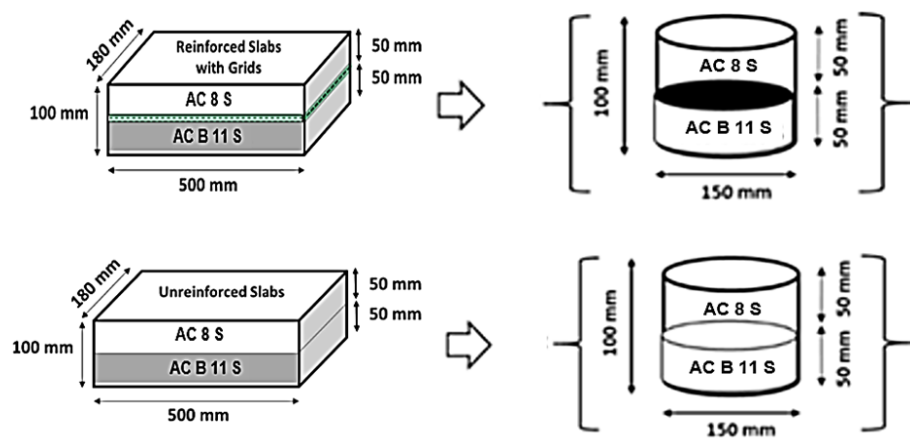


Figure 47: Advance Shear Test Specimen Production



Figure 48: Laboratory Produced AST Specimens (Dry and Wet)

The AST test was conducted on two groups of specimens i.e. dry and wet. For each group replicates of three specimens were tested. Wet specimens were conditioned according to the EN 12697-8 European standard. The AST test was conducted according to the following parameters. The details regarding all the tested specimens

with designations code are provided in Table 5. The specimens before testing are shown in Figure 48.

- Test Temperature: $20 \pm 5^{\circ}\text{C}$
- Rate of Loading: 50 mm/min
- Confinement/lateral Pressure: 0.6 Mpa
- Cylindrical Specimen size: 150 mm in diameter and 100 mm in height
- Conditioning Process: Dry & Wet (according to EN 12697-12)

Specimen type, designation, emulsion type details

Test	Slab Sample Designation	Core Sample Designation	Grid Type	Emulsion Type	Emulsion Dosage	Time (hr) Emulsion Breakage
Advance Shear Test (AST)	RE-1	Dry <u>RE-1</u> <u>RE-2</u> <u>RE-3</u>	No Grid	Cationic emulsion with a residual bitumen content of 60 %	3	200 g/m ²
	RE-2	Wet <u>RE-1</u> <u>RE-2</u> <u>RE-3</u>				
	AC-1	Dry <u>AC-1</u> <u>AC-2</u> <u>AC-3</u>	Product C	-	-	-
	AC-2	Wet <u>AC-1</u> <u>AC-2</u> <u>AC-3</u>				
	AA-1	Dry <u>AA-1</u> <u>AA-2</u> <u>AA-3</u>	Product A	Cationic emulsion with a residual bitumen content of 60 %	3	180 g/m ²
	AA-2	Wet <u>AA-1</u> <u>AA-2</u> <u>AA-3</u>				
	AD-1	Dry <u>AD-1</u> <u>AD-2</u> <u>AD-3</u>	Product D	Cationic emulsion with a residual bitumen content of 60 %	3	250 g/m ²
	AD-2	Wet <u>AD-1</u> <u>AD-2</u> <u>AD-3</u>				
	AB-1	Dry <u>AB-1</u> <u>AB-2</u> <u>AB-3</u>	Product B	Cationic emulsion with a residual bitumen content of 60 %	3	480 g/m ²
	AB-2	Wet <u>AB-1</u> <u>AB-2</u> <u>AB-3</u>				

Table 5: Specimen designation for AST

6.1.2.5 Results

The major findings of the AST test results shown in Table 6 reveals that in dry conditions, the reference specimens RE-D have on average the highest shear force value of 28.8 kN. The specimen AD-D among the reinforced specimens has on average the highest value of 27.1 kN in dry condition followed by un-reinforced reference specimens.

The AC-D-1 specimens registered 19.1 kN making the average value of AC-D as 17.4 kN and followed by the specimen AB-D with an average value 15.4 kN. It was noticed that the specimen AD-D-3 in dry condition and AD-W-2 and AD-W-3 during the conditioning process (water bath 22 °C) was broken at the interface before testing and therefore the results of these specimens are not available.

Advance shear test results

Sample	Fmax (kN)	dL(Fmax) (mm)	Average	St. dev.
Dry	AD-D-1	10.4	14.7	6.1
	AD-D-2	19.1		
Wet	AD-W-1	16.4	16.4	-
Dry	AA-D-1	27.8	27.1	0.7
	AA-D-2	27.1		
	AA-D-3	26.4		
Wet	AA-W-1	20.0	22.8	2.9
	AA-W-2	25.8		
	AA-W-3	22.8		
Dry	AC-D-1	19.1	17.4	1.5
	AC-D-2	16.3		
	AC-D-3	16.8		
Wet	AC-W-1	14.4	18.3	4.0
	AC-W-2	18.1		
	AC-W-3	22.4		
Dry	AB-D-1	13.6	15.4	2.2
	AB-D-2	17.9		
	AB-D-3	14.8		
Wet	AB-W-1	15.7	15.5	0.3
	AB-W-2	15.6		
	AB-W-3	15.1		
Dry	RE-D-1	31.3	28.8	3.0
	RE-D-2	29.7		
	RE-D-3	25.4		
Wet	RE-W-1	23.9	22.3	1.4
	RE-W-2	21.3		
	RE-W-3	21.7		

Table 6: Details of AST test results

As already mentioned, the AD-D-3 & AD-W-3 specimen were broken at the interface during the installation in the AST test machine. Also, the results show that specimen

AD-D-1 has a very low value of 10.4 kN in comparison with the value 19.1 for the specimens AD-D-2. Thus, the results of specimen product AD both in Dry & Wet presented in the analysis are now only based on the values of AD-D-1, AD-D-2 and AD-W-1. Upon moisture conditioning, the values of reference specimen, AA and AD reduces, which shows the effect of moisture damage. However, the values of specimen AC and AB slightly increases with moisture conditioning. This shows that moisture damage has no adverse effects on the specimens AC and AB. This could be related to the interface conditions as the AC reinforcement is made of a grid patch which therefore resisted the moisture effects. The AB grid is composed of grid with fabric, and this might have increased the chance of resisting the interface damage due to water. Overall, the results show mostly the same trend as evidenced from the results of the standard Leutner test.

The average AST maximum shear force results of specimens tested in dry conditions are shown in Figure 49. Both the reference RE and AA specimens (in dry condition) on average have the highest values followed by AC and AB in dry conditions. The specimen AD (in dry condition) has on average the lowest values which was expected due to the poor interface conditions as evidenced. The average results of specimens in wet conditions are shown in Figure 50. The results follow a similar trend as evidenced for dry condition specimen. However, the shear force values lowered respectively due to moisture damage effects. The comparison of dry and wet results is shown in Figure 51. Furthermore, after water conditioning specimen AA has higher resistance against the interface fracture compared to other reinforced specimens.

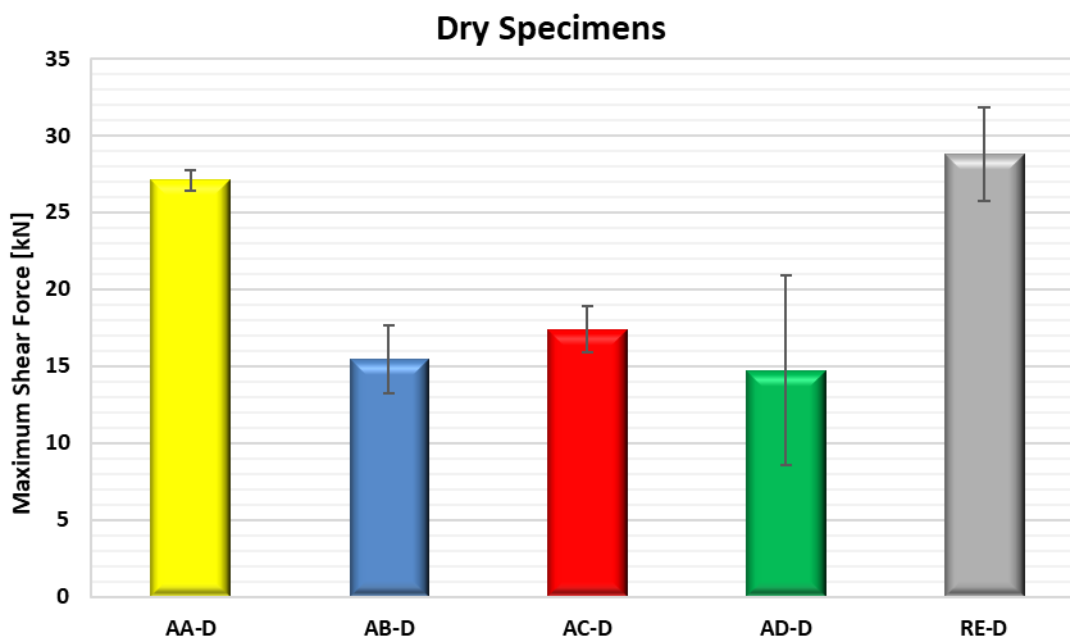


Figure 49: AST Maximum Shear Force of Dry Tested Specimens

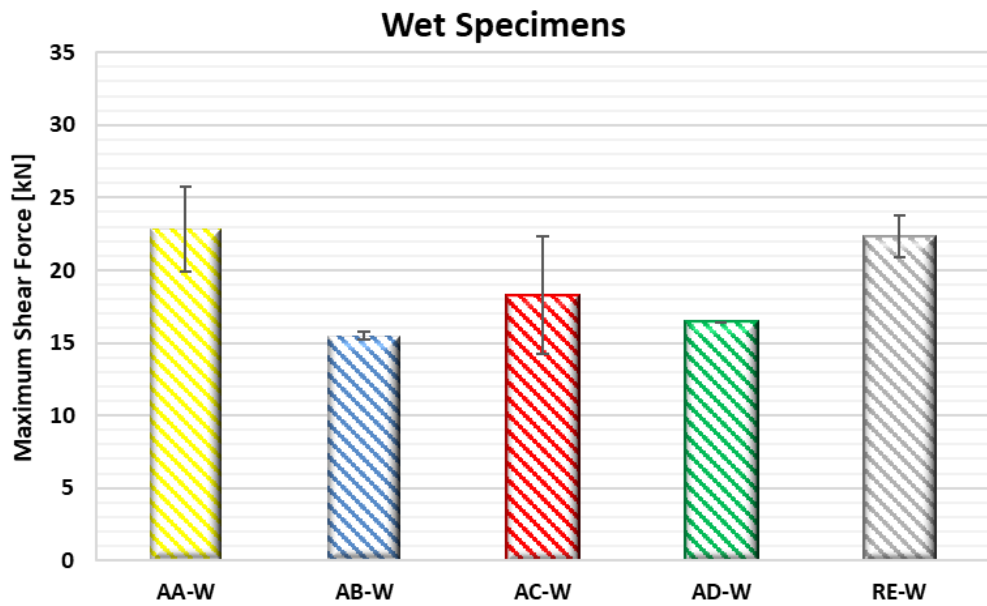


Figure 50: AST Maximum Shear Force of Wet Tested Specimens

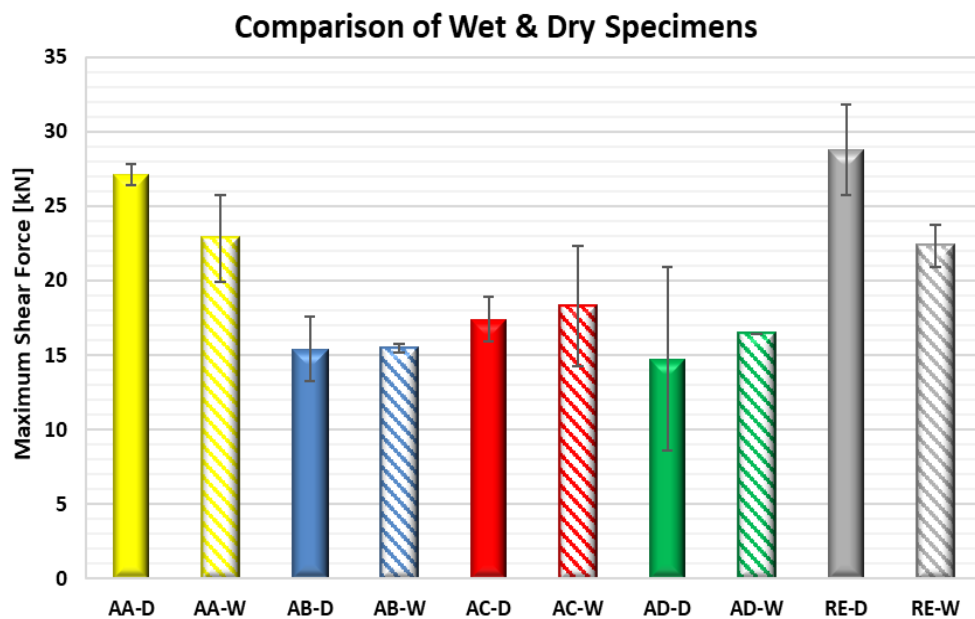


Figure 51: AST Maximum Shear Force, Comparison of Dry & Wet Tested Specimens

6.2 Crack propagation

6.2.1 Cyclic loading on beam specimens

The tests were conducted with the help of a servo-hydraulic dynamic machine as shown in Figure 26. The displacement of the piston is used as the feedback signal for the tests being performed. Dynamic loading and unloading cycles were applied. A load cell is

used to determine the applied force (P). The test temperature was maintained through mounting a conditioning chamber on the dynamic machine as shown in Figure 4.1. A thermal analog was placed near the test specimens to continuously record the temperature inside the conditioning chamber. The following test parameters were used to perform the crack propagation test.

Test Temperature: 5 °C

Loading sequence:

- Phase 1: Specimen's setting in the loading frame: Loading & unloading at $v = 1$ mm/s
- Phase 2: Crack initiation (high energy accumulation in the sample): Loading at $v = 0.5$ mm/s, unloading at $v = 1$ mm/s
- Phase 3: Crack advancement (lower energy in the sample): Loading at $v = 0.5$ mm/s, unloading at $v = 1$ mm/s
- Phase 4: Crack propagation: Loading & unloading at $v = 1$ mm/s

6.2.1.1 Loading sequence

Crack propagation laboratory test setup is shown in Figure 52. The loading setup of the crack propagation test and the dimensions of the beam are shown in Figure 53. The test was executed in 4 different phases according to the loading sequence as shown in Figure 54. During phase 1, three cycles with a loading and unloading rate of 1 mm/min were maintained to establish proper contact of the specimens with the loading frame. In phase 2, crack initiation was started by applying a continuous load at a rate of 0.5 mm/min (high energy accumulation in the sample) and the loading was removed at a rate of 1 mm/min when the crack started to initiate from the tip of the notch. The crack advancement above the notch was followed in Phase 3 by applying the load at a rate of 0.5 mm/min (lower energy in the sample) until the crack reached the desired length (i.e. below the interface) and the unloading of the specimen was executed at the rate of 1 mm/min. The crack advancement step in Phase 3 of the experiment was considered as a key step to shorten the overall test duration. Finally, in phase 4, crack propagation was initiated by applying a cyclic loading & unloading at rate of 1 mm/min.

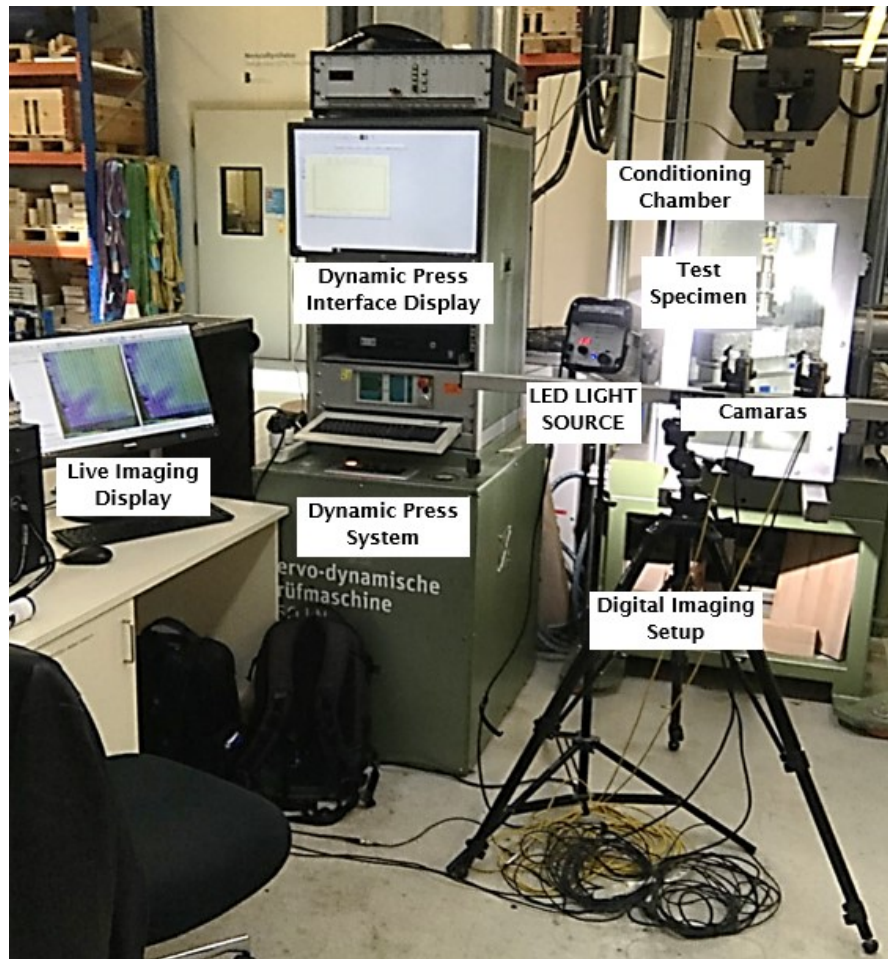


Figure 52: Crack Propagation Laboratory test setup

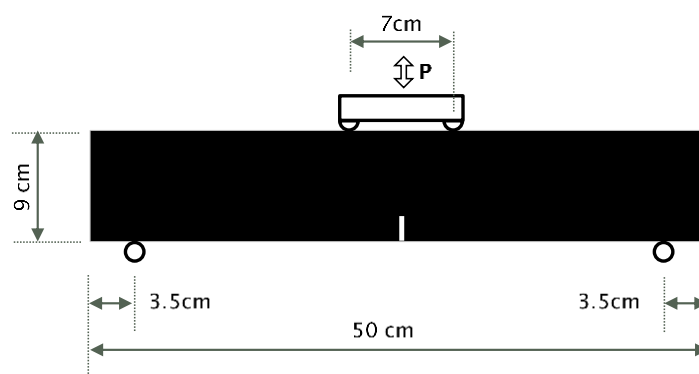


Figure 53: Schematic of 4-point bending beam test setup

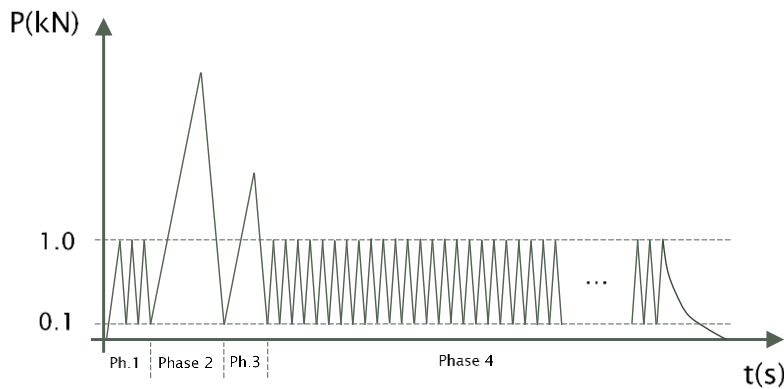


Figure 54: Loading sequence of test setup

6.2.1.2 Crack propagation monitoring

The front face of the test beam specimen is painted in white, and a light source is installed to focus the area of interest where the crack path is likely to appear. The whole process of the test execution was monitored, and images were recorded by using a digital image correlation (DIC) camera setup (cf. Figure 52). Phases 1, 2, and 3 of the loading sequences were monitored by capturing an image with a frequency of 1 Hz, i.e. capturing an image after every second. However, the final phase 4 was recorded by capturing an image after every 300 seconds. The inspection of the images allowed the visualization of the crack and following its propagation path during the test.

Since the asphalt mixtures are known as viscoelastic materials, the crack propagation test is defined as more sensitive to the testing temperature. Under low temperature conditions (e.g. at 5 °C), the asphalt mixture behaves more elastic in nature and ultimately provides a brittle response under different loading conditions. Also, it is known that asphalt at low temperature is more susceptible to cracking (Nguyen et al., 2016). Therefore, the current test scheme represents the test results of crack propagation test in low temperature conditions.

In the crack propagation test setup, the execution of crack was carefully controlled manually using an interface with the test program setup developed specifically for crack propagation. During the initiation process of Phase 2, the loading curve is carefully followed to allow the load to reach its peak. Subsequently, the load is then immediately removed, and the next level of Phase 3 is started, where the load is applied again at the same rate to further advance the crack propagation. However, during this process, some of the specimens are damaged due to the crack movement charged above the interface during Phase 3 of the experiment. Therefore, the damage within the specimen is usually accounted by not exactly following the crack advancement step where the crack must be kept below the interface. Even though this damage is not really recognized during execution of the test. However, by post analyzing the images captured during the test execution process, the specimens experiencing the crack advancement at around or even above the interface are then declared as damaged. Furthermore, the results of these specimens in terms of the number cycles reaching until failure have also significantly lower numbers compared to the same type of specimen that was tested without any damage. Therefore, the results of these damaged specimens are discarded and not included in the analysis part.

For the investigation of the crack propagation patterns and mechanisms of grid-reinforced asphalt concrete, a four-point bending beam (cf. Figure 55) test was carried

out on reinforced double-layered beam specimens [8]. In this research, four grid types, two mixture types (AC S 8 for top layer and AC S 11 for bottom layer), and tack coat of dosage depending on grid type was used to evaluate the effects of different grid reinforcements in comparison with non-reinforced specimens. All the tests were carried out at a test temperature of 5 ± 3 °C.



Figure 55: Test specimen mounted over 4PB bending beam loading assembly

The image processing method has been widely used as a nondestructive system to evaluate 2D or 3D geometry in numerous scientific fields. In the field of civil engineering, image processing has been successfully applied in multiple applications such as pavement distress assessment, site evaluation using satellite imaging, and analysis of crack propagation and microstructures in cement-based materials. Related to asphalt pavement materials, several computer-vision based systems have been developed, which are broadly categorized into field assessment and laboratory applications. Concerning field assessment, algorithms were developed for the identification of pavement distress type and magnitude. Nonetheless, these algorithms also allow identifying the fractured surfaces of asphalt mixtures such as broken aggregates or the adhesion and cohesion failures at the interface of failed specimens. In this work, the main concern regarding imaging technique is to quantify the crack propagation length via image processing on asphalt beam specimens fractured by using a four-point bending beam cyclic loading test. The aim was to identify the crack propagation advancement position e.g. crack length above the tip of the notch and below the interface. An example of the specimen image where the advancement of the crack is visible is shown in Figure 56. As a proposed procedure pertaining to image processing, localized thresholding was implemented by dividing the image into smaller blocks and identifying a local threshold and finding the crack pixels using the threshold of each block. Correlating the intensity and the relative values of the RGB/Grayscale components of the image, the region of interest was extracted from the sample image

and its subsequent crack analysis was performed. The image is converted into a black and white image in the end to identify the existing cracks.

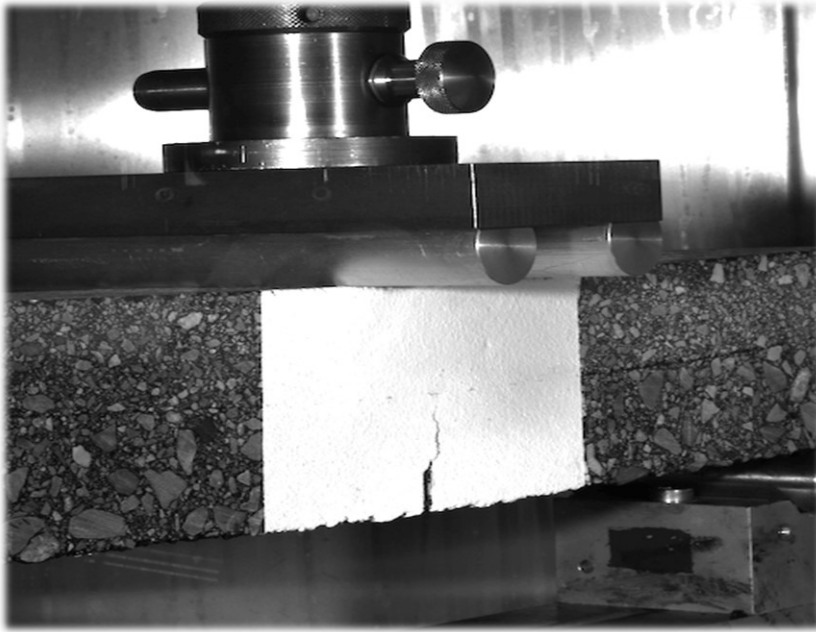


Figure 56: Sample image before analysis

The proposed method was implemented in 3 stages i.e., Background separation, region of interest [ROI] enhancement, and crack length computation. The background separation was executed using the foreground mask by performing a subtraction between the current frame comprising the crack [ROI] and the background model comprising the unwanted region. The next phase was the localized thresholding where contrasting was done by varying the values of pixels respectively, depending upon the local threshold, of the blocks to obtain the ROI. To enhance the quality of ROI, thinning of image was done which was then followed by denoising to remove the unwanted pixels. In the final phase, the crack length was calculated using the coordinate-based pixel computation. The sample image post contrast adjustment implementation is shown below in Figure 57.

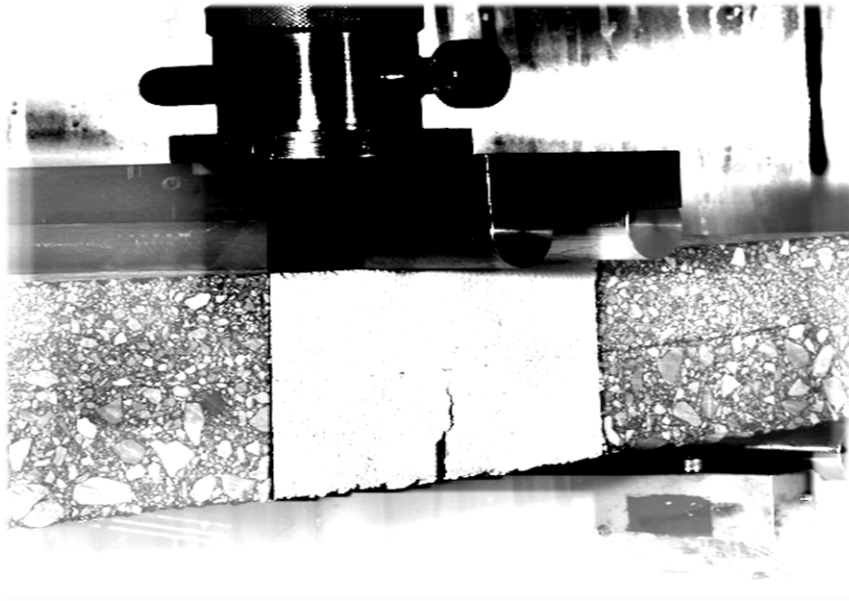


Figure 57: Image sample post SD and Contrast adjustment

The next stage was the localized thresholding. Post contrast adjustment, the threshold binary image is shown below in Figure 58. This stage segregates the cracked pixels from the other pixels by increasing the intensity of the other pixels and decreasing the intensity of the cracked pixels i.e., ROI encircled in red.

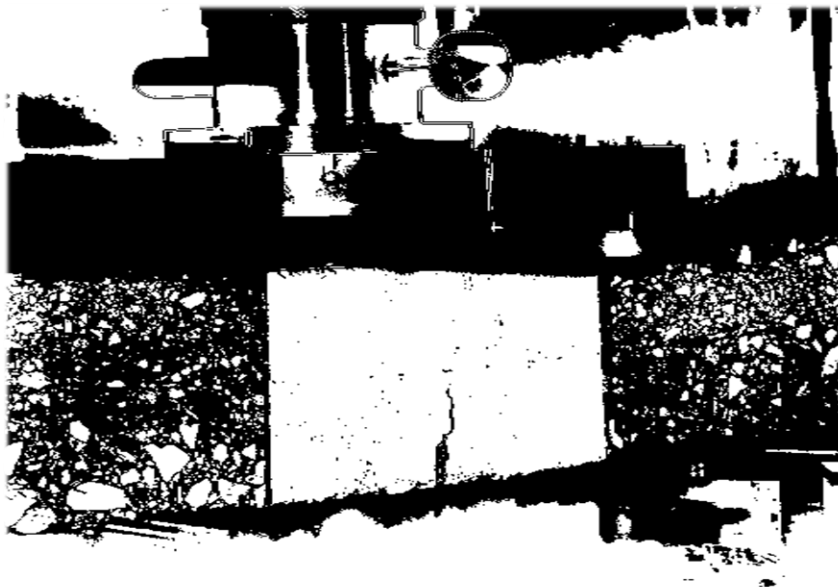


Figure 58: Thresholded binary image

Post contrasting, the image was then converted into binary image depending on the average value of region of interest. Morphological operation and noise reduction is then performed to verify the neighborhood of pixels i.e., for assessing the non-zero pixels in the neighborhood.

To improve the performance of the proposed method, it was implemented via C++ programming language by importing Open Computer Vision (OpenCV) library, which

was used for image processing functions. It is to be noted that in the proposed work there are three batches of images whereby each batch size is approximately 2.5 Gigabytes or less. In this regard, the significance of the intense parallel computation of images at different cycles will cut down the execution time. There are numerous application programming interfaces (API) available for parallel processing; however, for the proposed experiment data, Open Multi-Processing (OpenMP) for parallelizing the execution has been used. OpenMP is an API for multiprocessing programming which supports shared memory architecture. The prime feature of OpenMP is its vast instruction set i.e., OpenMP pragmas which is used for auto parallelization. For the proposed experiments, OpenMP pragma (omp) was used to parallelize the intensive matrix computation involved in standard deviation and localized thresholding computation.

The complete proposed procedure is described in the block diagram as shown below in Figure 59.

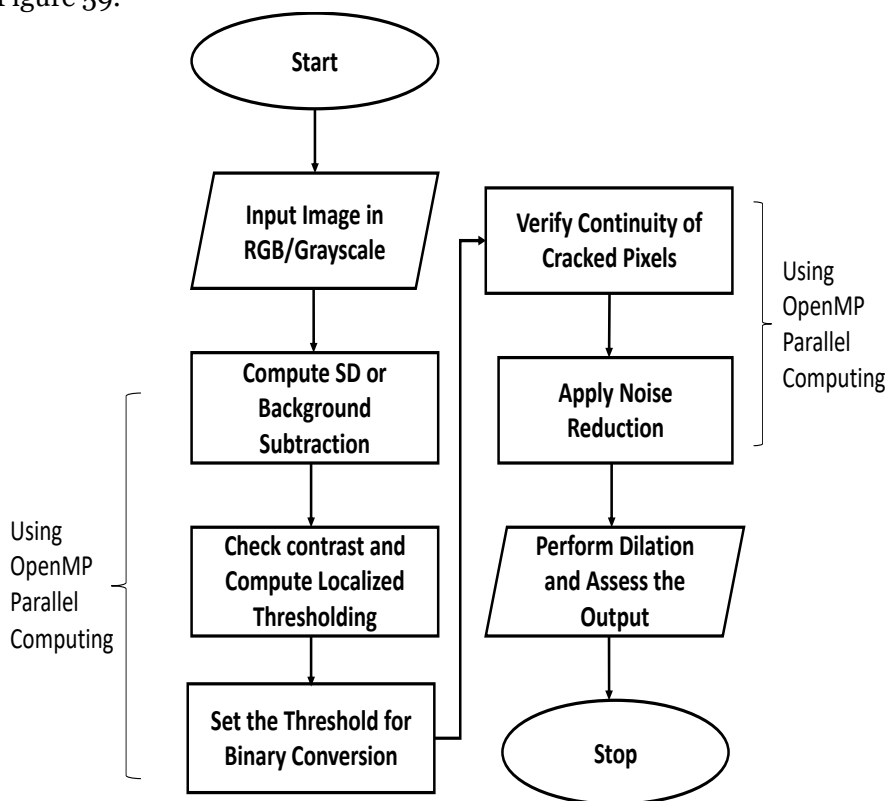


Figure 59: Flowchart of the proposed methodology

6.2.1.3 Beam specimens preparation with continuous and overlap grids

Two beam specimens were produced from each of the double layer slabs by cutting it into two halves. It is worth noting that the reinforced grids laid over the slab should have an equal/even numbers of stands across the width and length of the beam. So as upon cutting into two halves, each beam has an equal number of grids strands. To ensure firm contact of loading strips with the top surface of the beam, 1 cm of the top layer beam is also cut using a diamond saw cutter. The beams with a final dimension of 50 x 9 x 9 cm (L X W X H) were used for the crack propagation test. Furthermore, to facilitate and provide proper guidance for the crack propagation, a notch of 15 mm

depth and 3.0 mm width was induced at a length of 250 mm in the middle of the bottom slab. The double layer beam produced from a slab specimen is shown in Figure 60.

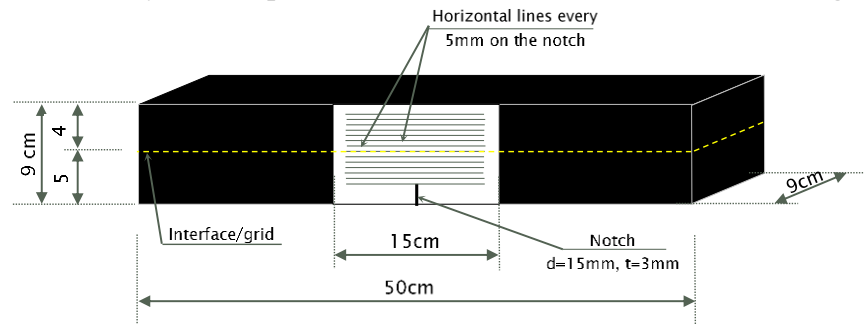


Figure 60: Schematic of Beam taken out from Double Layer Slab Specimen

The beam specimens for the investigation of the effect of grid overlap were produced by following the same procedure used for continuous length grid samples except that the grid was laid at the interface with an interruption having an overlap of 15 cm lengths. Figure 61 shows a schematic example of the overlap length grid specimen prepared for the crack propagation test. During the installation of the grid, special care was considered for the alignment of the grid stands. This means that the overlap grids are positioned in such a way that both upper and lower stands of the overlap grids are aligned exactly on top of each other. The overlap length of the grid is taken from the middle of the notch, for example a 15 cm grid overlap length means that the overlap portion of the grid is positioned 7.5 cm from each side of the middle of the notch.

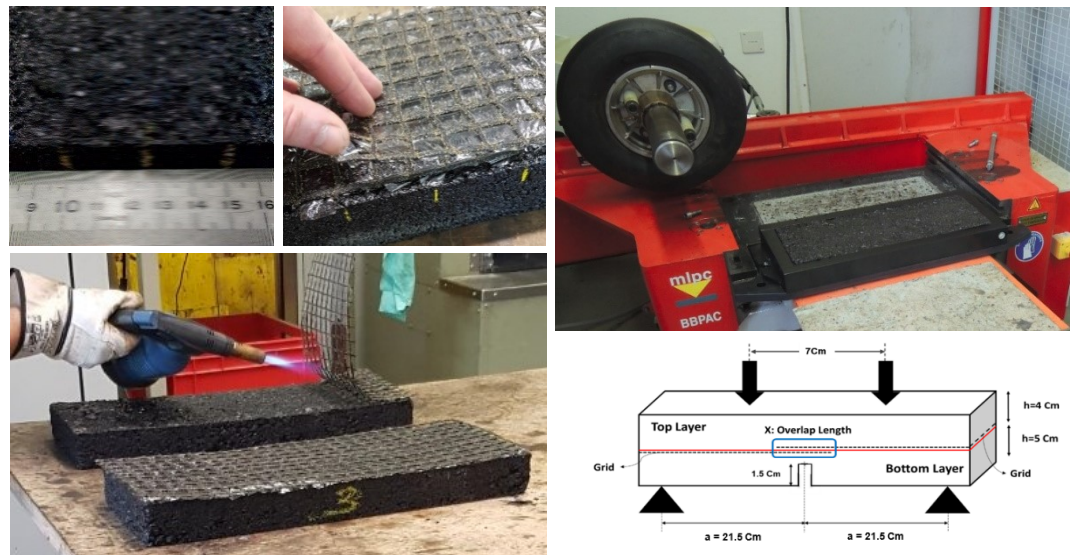


Figure 61: Overlap Length Double Layer Slab Specimen and Schematic of Overlap Length Beam

6.2.1.4 Results: analysis of force vs time

Figure 62 shows the time (X-axis) vs force (Y-axis) diagram, where all the 4 phases of the test can be clearly seen. The first phase of the test shows three loading cycles up to maximum load of 1 kN is applied for the establishment of firm contact between specimen and the loading strips. In the second phase, the specimen Ref_1 has reached a maximum of 3.34 kN load in order to induce a crack above the notch tip.

Furthermore, in the third phase the load is again applied to advance the crack below the interface. As already mentioned, the third phase of the test is performed due to reducing the overall total duration of the test until failure along with a reduced accumulation of the energy within the specimen. In parallel to this, it will also allow to identify a common point of interest for the evaluation of crack propagation across the interface in the presence of grid reinforcement. Finally, the cyclic loading of 1kN magnitude is continuously applied to propagate the crack through the interface until complete failure of the test specimen. The time versus force results of all the reinforced tested specimens given in Annex 11.3.1 showed that the maximum load needed to initiate the crack in phase 2 of the test does not follow any specific trend. Although, for few of the specimens, the load required to initiate crack above the notch tip was even lower comparatively and the reason behind this was unknown. However, for some of the specimen e.g. Or-A-2, the maximum load remained around 4.51 kN.

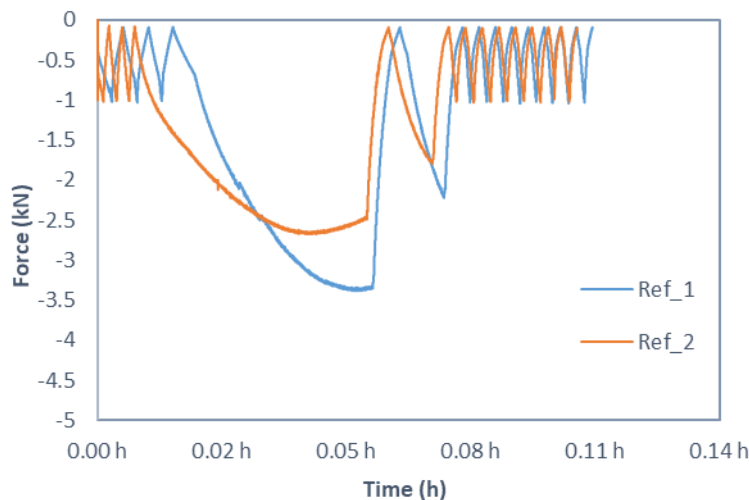


Figure 62: Force vs Time diagram for Ref Specimen

6.2.1.5 Results: cyclic analysis full length grid

The test results of continues reinforced and unreinforced specimens using 4-point bending beam crack propagation are presented in Table 7. The exact dimension e.g. beam size, notch size and dry weight of each beam specimens were recorded before the commencement of test. In the Table 7, the total number of cycles (T) are defined as the number of cycles required by a beam specimen from the test execution until reaching the complete failure. However, based on the execution of the test, this number of cycles needs to be adjusted to compare the results among different beam specimens. This is because since the crack initiation length of each individual beam specimen during crack initiation and advancement phase (i.e. phase 2 and phase 3) cannot be controlled to be exactly similar all the time. Therefore, it is important to know the actual number of cycles needed to propagate the crack from one common reference point until complete failure of the specimen in terms of comparing different specimens.

To calculate the adjustment of number of cycles, a specific length of the crack above the tip of the notch (Phase 3) is first needed for every individual beam test specimen that are going to be compared. Therefore, with the help of image analysis technique, first the specific crack length (SCL) for each individual specimen is identified. Then,

the maximum value of SCL reached by any of the specimen is defined as a reference point. Next, as shown in Figure 63, the maximum value of the SCL (reference) is taken to identify the same value among the other set of specimens that are going to be analyzed comparatively. Further, the specific number of cycles (SNC) are then obtained against the respective SCL of each individual specimen. Finally, the total adjusted number of cycles (ANC) are then calculated based on the total number of cycles (T) minus specific number of cycles (SNC) within each specimen. The details of results based on total ANC derived from Image analysis are represented in Table 7.

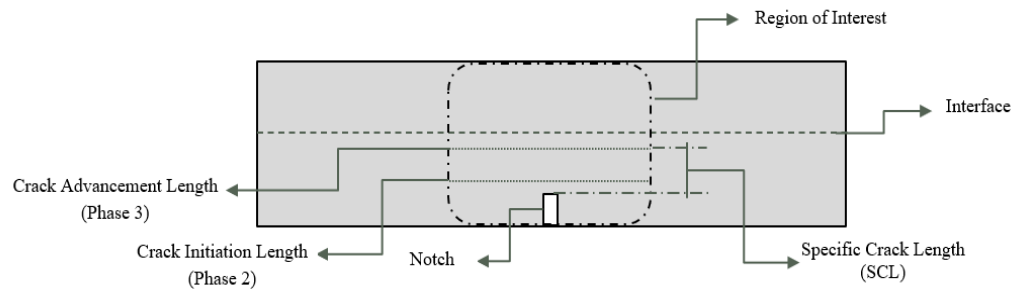


Figure 63: Schematic diagram of sample identifying SCL

Full-length grid beam specimen geometry and test results of crack propagation

Sr. #	Specimen	Dry Wt. of Beam (g)	No. of Cycles (T) [-]	Adjusted No. Of Cycles [-]
1	Ref-1	8642	618	615
2	Ref-2	8546	344	322
3	Cr-B-1	8860	1835	1832
4	Cr-B-2	8702	1731	1738
5	Cr-A-1	8815	1238	1235
6	Cr-A-2	8846	1067	1064
7	Cr-C-1	8778	1391	1388
8	Cr-C-2	8636	1155	1152
9	Cr-D-1	8946	1467	1464
10	Cr-D-2	8866	2001	1998

Table 7: Full-length gird beam specimen geometry and total number of cycles until failure

Figure 64 shows the results of displacement vs total number of cycles until failure. It is worth mentioning that the adjusted number of cycles that are derived based on the individual specimen crack initiation and advancement with reference to the specific number of cycles are represented in Figure 66. However, the results shown in Figure 64 indicate the individual performance of all the tested specimens with and without continuous grid reinforcement.

A DIC image of unreinforced and reinforced specimen in Figure 65 shows the crack propagation pattern. It is evidenced that the crack propagates vertically in unreinforced specimen compared to reinforced specimen where the crack has both

vertical and lateral propagation. As shown in Figure 66 the different types of reinforcement grids used in this study demonstrated that the total number of cycles needed to fracture the specimens with full length grid has reached up to more than 2,000 cycles. This shows the prominent effect of the reinforcement grids compared to the specimens without reinforcement grid. The results show that specimen Cr-D-2 has registered the highest number of cycles i.e. 2001 compared with the non-reinforcement specimen and all other reinforcement systems. However, the repeatability of Cr-D-2 with Cr-D-1 was found to be low. This can be explained by the interface conditions of grid reinforcement and its individual materials property that has offered higher resistance against the crack propagation and provided a larger delay in terms of crack propagation. The variability with specimens of Cr-A-1 & Cr-A-2 and Cr-B-1 & Cr-B-2 was found to be good. Overall, all the reinforced specimens achieved a higher number of cycles compared to the unreinforced specimens. It is worth noting that in specimen Cr-B-2 and Cr-C-1 there is an abrupt failure in comparison with Cr-B-2 and Cr-C-1 respectively. This can be directly related to the grid interlock position at the interface during crack propagation.

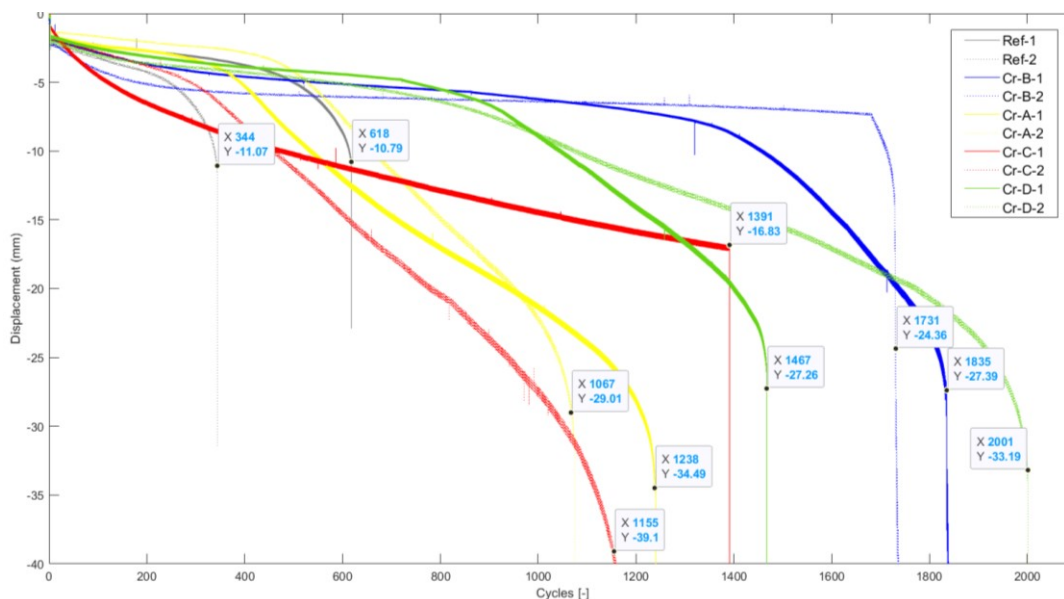


Figure 64: Displacement vs cycles (reinforced full length grids)

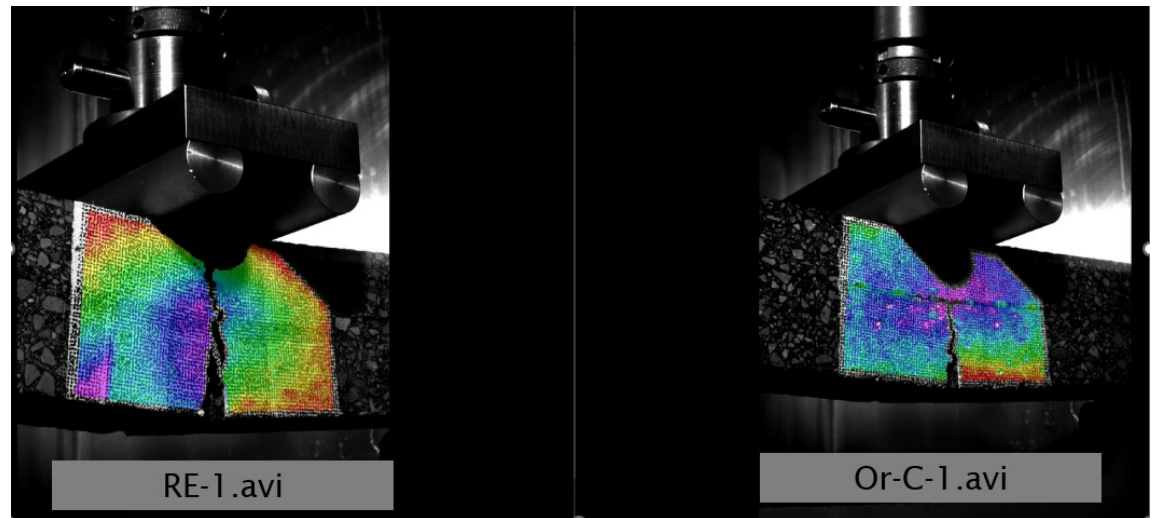


Figure 65: DIC image of unreinforced vs reinforced specimen crack propagation

The adjusted number of cycles calculation based on the crack propagation from a common reference point below the interface until the complete failure of the specimen (cf. Figure 63) shows the significant effect of grid reinforcement compared to reference specimen. Based on the image analysis results, the SCL during the advancement phase (Phase 3) of the crack propagation test is measured as 6.3 mm above the tip of the notch and taken as a reference point. Next, based on this reference point, the adjusted number of cycles are calculated for each individual test specimen and represented in Table 7. It can also be observed that different types of grids had different resistances against crack propagation. Furthermore, it was observed that the overall trend between different grid reinforced specimens using adjusted number of cycles remains similar when compared with the results of number of cycles obtained without adjustment.

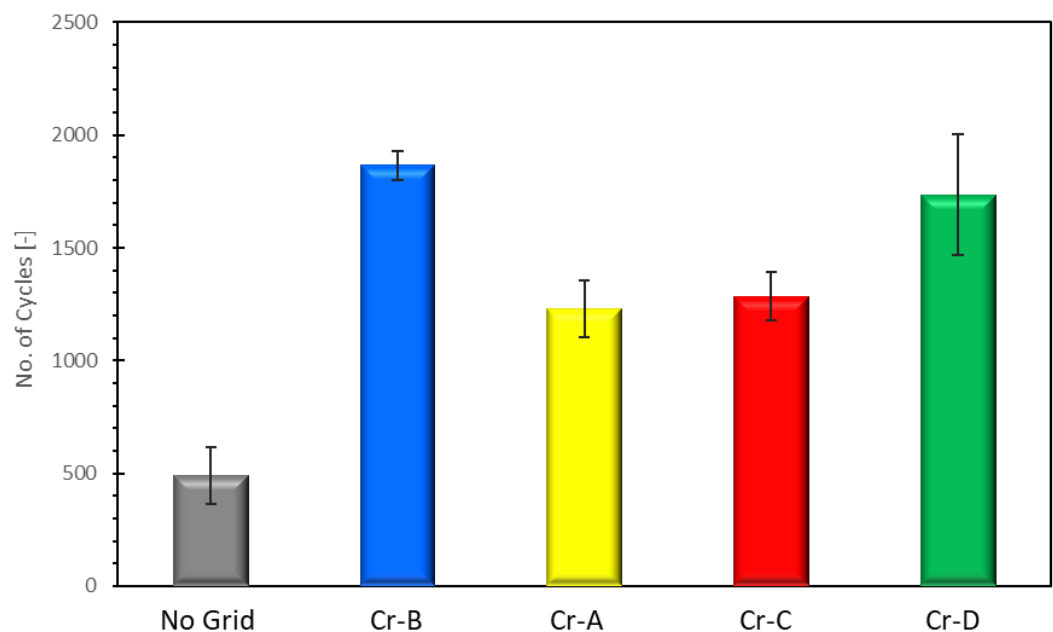


Figure 66: Comparison of reinforced full length grid samples in terms of cycles

6.2.1.6 Results: overlap length grid

The overlap length of a reinforcement grid is an important aspect from the construction point of view. In the field, the overlapping of grid during laying is usually provided due to the practical limitation of the grid size and laying technique. However, this is very important when it comes to the requirements for different applications and, for example, different road geometries. Also, the limitation due to laying devices availability play an important role in the installation process of different grid sizes. Hence, the overlapping of a reinforcement grid is a key aspect to be taken as an important characteristic of the investigation. It will be interesting to investigate the overall crack resistance performance of the same type of grids used for reinforcement but this time with an overlap length. Based on supplier experience and recommendation, the overlap length of 15 cm was selected for all test specimens. The following section provides detailed results and discussion of the specimens tested with grid reinforcement comprised of 15 cm overlap lengths.

The details regarding overlap length grid reinforced beam specimens and results based on total number of cycles obtained from crack propagation test are presented in Table 8. The results of displacement vs total number of cycles until failure for overlap length test specimens are shown in Figure 67. These results are based on the total number cycles obtained by each individual specimen until its complete failure. The image analysis results showed that the SCL during the advancement phase (Phase 3) of the overlap test specimens is measured as 6.5 mm above the tip of the notch and taken as a reference point. The adjusted number of cycles that are derived based on the individual specimen crack initiation and advancement with reference to the SNC are represented in Figure 68. The results shown in Figure 68 indicate the individual performance of all the tested specimens with overlap length of 15 cm grid reinforcement. Like the results of full-length grid reinforce specimens, it is noted that the repeatability within the results for some of overlap length test specimens was not very good. This might be due to the presence of grid reinforcement. That is for example, at the interface position in the case of specimen Or-C-1 as shown in Figure 67, where it was observed that the crack propagates horizontally by pulling away the bottom part of the beam (i.e., below the interface). This indicates the limitation of the test at laboratory scale; however, such situations cannot happen in the field on a real scale. Overall, in the overlap test results, it was evidenced that the different types of grid reinforcement had different effects. Furthermore, the effect of overlap length of grid reinforcement depending on the type of grid has an influence against the crack resistance. The detailed description on the comparative performance of the overlap length with full-length reference test specimens is given in Figure 69 and is based on adjusted number of cycles obtained via image analysis.

The overlap length crack propagation test results in Figure 67 shows the number of cycles on x-axis and the corresponding deformation on y-axis. The test results show that the specimens Or-B-1 and Or-B-2 reached the maximum number of cycles 3804 and 3877, respectively. The specimens Or-A-1 and Or-A-2 survived 2056 and 1502, respectively. The specimens Or-C-1 and Or-C-2 have reached over 2000 cycles. However, the specimen Or-D-1 and Or-D-2 have very low numbers of cycles i.e. on average a lower number of cycles compared to reference specimens without grid reinforcement. The poor performance of specimens Or-D-1 and Or-D-2 is directly related to the bonding condition of the interface. The same effects can also be observed during the AST layer adhesion test, where the specimens broke apart even before the

test commencement. This effect can be overcome by improving the grid installation conditions and layer adhesion situation.

Overlap-length grid beam specimen geometry and test results of crack propagation

Sr. #	Specimen	Dry Wt. of Beam (g)	No. of Cycles (T) [-]	Adjusted No. of Cycles [-]
1	Ref-1	8642	618	615
2	Ref-2	8546	344	322
3	Cr-B-1	8483	3804	3801
4	Cr-B-2	8678	3877	3874
5	Cr-A-1	8815	2056	2053
6	Cr-A-2	8846	1502	1499
7	Cr-C-1	8621	2260	2557
8	Cr-C-2	8581	2365	2362
9	Cr-D-1	8946	354	351
10	Cr-D-2	8866	69	69

Table 8: Overlap-length grid beam specimen geometry and total number of cycles until failure

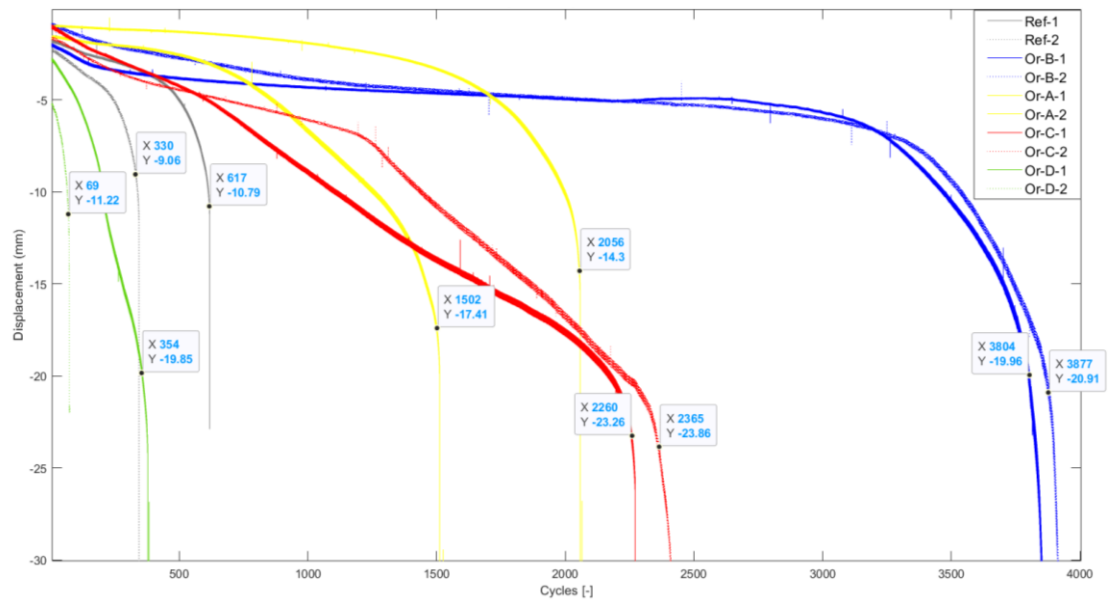


Figure 67: Displacement vs cycles (reinforced overlap length grids)

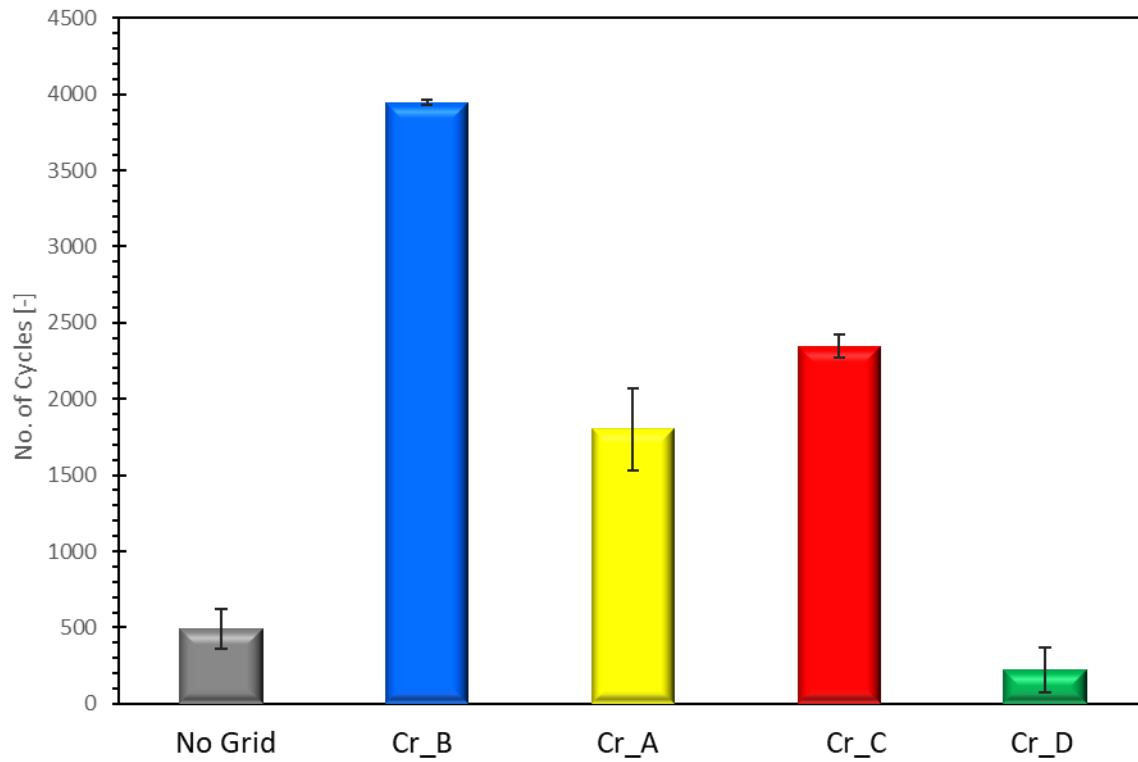


Figure 68: Comparison of reinforced overlap length grid samples in terms of cycles

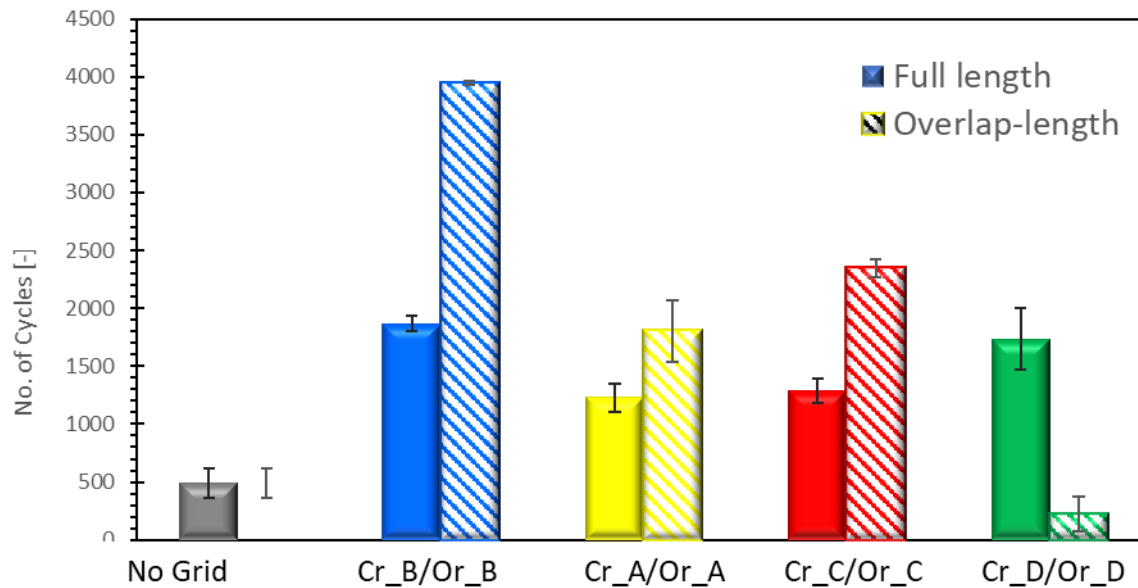


Figure 69: Comparison of reinforced full and overlap length grid samples in terms of cycles

6.2.2 Traffic loading on slab specimens (MMLS3 tests)

6.2.2.1 Description of the Mobile Model Load Simulator MMLS3

The MMLS3 (shown in Figure 70) is a laboratory-scaled accelerated pavement loading device used for pavement trafficking, simulating real pavement loading conditions. The machine is intended to induce pavement distress in a short period of time under the loading of repetitive rolling tires. It consists of a rigid steel frame with four adjustable feet and applies a downscaled load with four single pneumatic tires that simulates traffic, moving in one direction on a circulating belt like a chain saw. Each tire has a diameter of 0.3 m and a width of ca. 0.10 m and loads the pavement through a spring suspension system over a length of 1.2 m. The tires pressure can be adjusted between 560...800 kPa. The device can be moved laterally during loading to simulate the wandering of the tires in real pavement, or it can be fixed in one position to simulate channelized loading (i.e. the tires apply the loads following the same wheel path). The machine can run at a maximum load of 2.5 kN and a speed of 9 km/h, allowing a maximum of approximately 7200 load applications per hour. This corresponds to a loading frequency rate of nearly 2 Hz. The entire device has a length of 2400mm, a width of 600mm, a height of 1150 mm and a weight of 800 kg. Due to the reduced and limited weight, the machine can be moved around relatively easily. The MMLS3 can be used outside to evaluate real pavement in-situ, or in the laboratory to load slabs in a climatic chamber for precisely controlling the testing temperature. It is usually used to investigate the rutting performance of asphalt mixtures at high temperatures (up to 50 °C) or their crack susceptibility at lower temperatures (10 °C).

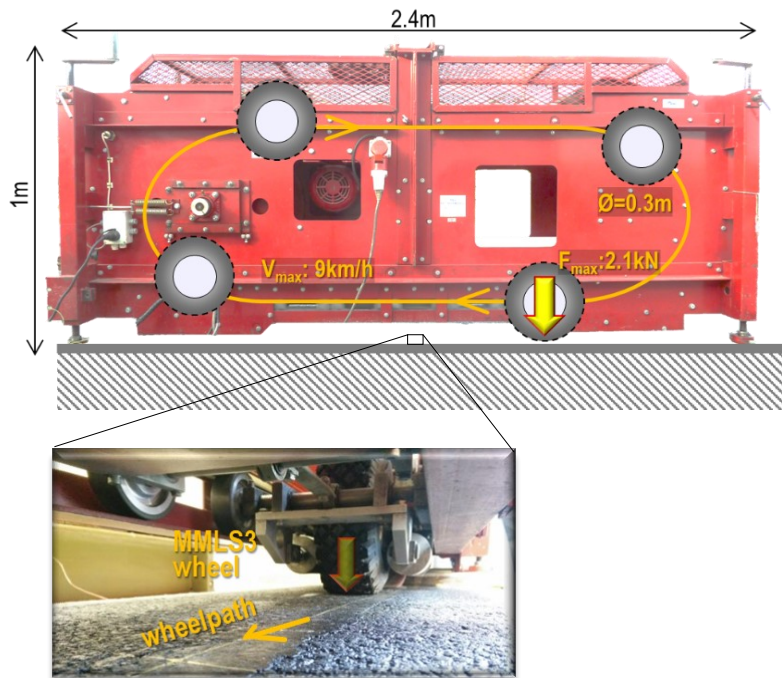


Figure 70: View of the MMLS3 with its 4 wheels and working principle

The advantage of using this type of testing machine over other small scale laboratory tests is the possibility to simulate the complex stress fields produced by vehicular loading in real pavements. As mentioned earlier, the MMLS3 can be used in-situ on existent roads or in combination with plate shaped specimens cut from a real pavement

or slabs compacted in the laboratory using a reduced scale roller compactor (see Figure 105). This allows the testing of more representative samples, compared to cylindrical Marshall, gyratory, beam or trapezoidal specimens.



Figure 71: Roller compactor and specimen production for the MMLS3

Overall, the MMLS3 bridges the gap between laboratory and in-situ performance of the materials and systems tested. Specifically, in the case of the grids, it is possible to construct laboratory slab-shaped specimens large enough to allow the installation of the different reinforcing systems in multi-layered systems. These types of specimens and loads permit testing to work in the same way as they do in situ, without local effects related to small specimen/grid size relationship. Furthermore, the tests of the different systems are performed under controlled loading and temperature conditions, allowing a confident comparison regarding their performance.

6.2.2.2 Experimental setup

With the aim of studying the crack propagation susceptibility of the different reinforcement systems, a two-layered slab setup was conceived to replicate a pavement superstructure. As for all the other tests carried out in this project, ACB11S for the base course and AC 8S was used for the surface course. As explained earlier, the mixtures 5.1 were heated in an oven, mixed and compacted using the medium size Empa rolling compactor to build the slabs. The different grids studied in this work were installed between the asphalt layers to provide a reinforcement capable of delaying cracks formation and propagation under the repetitive MMLS3 tire loading along the centre of the slab's length. The grid installation happened in the same way as shown earlier for the preparation of the interlayer bond test (Leutner). As in the case of the Leutner test specimens, in order to avoid errors and uncertainties, the different manufacturers were invited to Empa to install their reinforcement systems themselves, using their own material and equipment. The situation with no grid was studied for benchmarking the performance of the reinforcement grids. In this case, only cationic emulsion in a concentration of 200g/m² with a bitumen content of 60 % was applied with a brush as a tack coat.

In order to create a weakness in the slab as to facilitate the initiation of a crack, the design considered the preparation of a notch on the middle of the bottom layer,

perpendicular to the MMLS3 traffic wheel path on the surface of the slab. The specimen was positioned on top of a rubber mat to allow some bending of the slab under loading; in the same way a subgrade provides a relatively low stiffness support to the superstructure. Under these conditions, the bending should be able to generate a tension stress at the tip of the notch, without reaching the failure stress immediately but high enough to create a fatigue failure over several loading cycles. Furthermore, this bending movement would allow to activate the reinforcement function of the grids. The failure criteria used to terminate the test was, when a crack initiated on the tip of the notch reached the surface of the slab. An importance aspect to consider for the organization of the experiments was that each test would not take longer than 5 days, i.e. a working week starting on Monday and finishing at latest on Friday. This was an important condition to avoid stopping the tests during weekend and during night, as to prevent healing due to rest periods, initially, the size of the two-layered slab specimens was set to 1.6 m x 0.50 m and each layer having a thickness of 4 cm, making 8 cm the total thickness of the specimen. The notch depth was set to 3 cm and the thickness, determined by the thickness of the cutting blade, was 3 mm. The rubber mat used in a first instance had a thickness of 12 mm and a hardness of ca. 35° in the shore scale A. Three preliminary trials using a MMLS3 load of 2.1 kN and at a speed of 9 km/h (corresponding to a loading frequency of 2 Hz) shown that the stiffness of the system was too high (notice the deflections at the so the test lasted several weeks. Moreover, necessary stops over the weekends and sometimes to do maintenance in the MMLS3 were inducing healing, changing the conditions of the test for each experiment.

Therefore, several changes in the initial setup were defined with the aim of reducing the overall stiffness to increase the bending moment and the tension strains at the tip of the notch for limiting the testing time to a maximum of 5 days. The first change was to reduce the width of the slab from 0.50 m to 0.45 m. Further, the thickness of the top layer was reduced from 4 cm to 3.5 cm. With these two changes, the size of the slab specimen was set to 1.6 m x 0.45 m with a total thickness of 7.5 cm. Moreover, the rubber mat was changed for a thicker and softer mat. The new 2 cm thick elastomeric mat had a hardness of ca. 15° in the shore scale A, allowing larger deformations. Although the depth of the notch was not changed, a thicker 5 mm blade was used to saw the slab. The short edges of the slabs were placed on steel profiles (bearings or supports) to increase the bending moment under load and between them and below the slab, the mat was placed. The whole setup was fixed onto a stiff concrete plate to anchor the MMLS3 and it was then placed in a container at 20°C for a controlled loading temperature situation. An additional change relative to the previous trials was made on the loading conditions: instead of using 2.1 kN and a loading frequency of 2 Hz, the load was increased to 2.5 kN and the trafficking speed was reduced to 3.6 km/h which is equal to 1 Hz loading frequency. The aim of the speed reduction was to increase the effect of the load on the slab thanks to the viscoelastic properties of the asphalt. The crack formation and propagation were monitored using of LVDTs and Digital Image Correlation (DIC) cameras. A photo collection of the construction process and testing setup is shown in Figure 72.

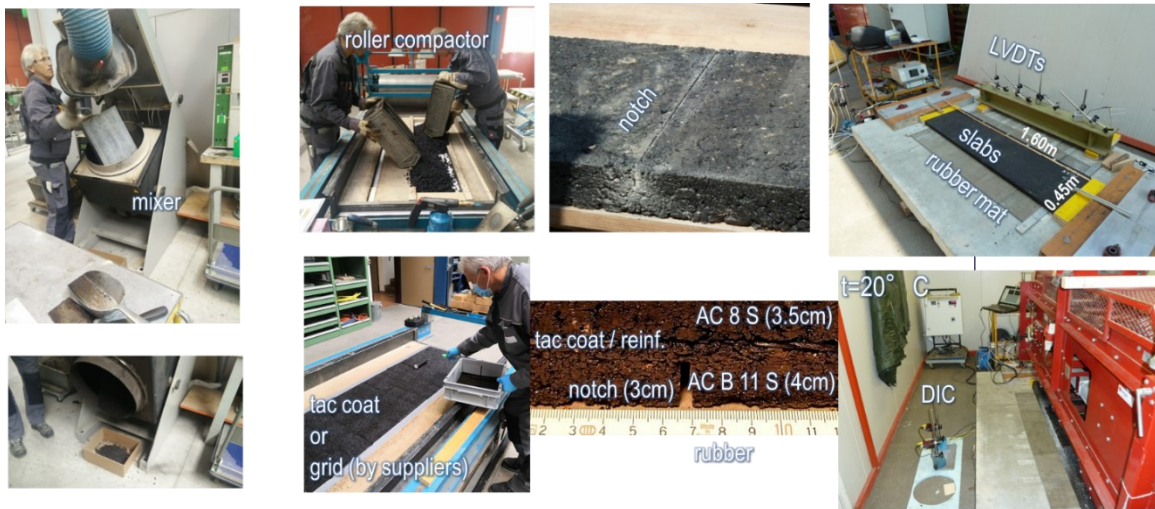


Figure 72: Photo collection of the test preparation process

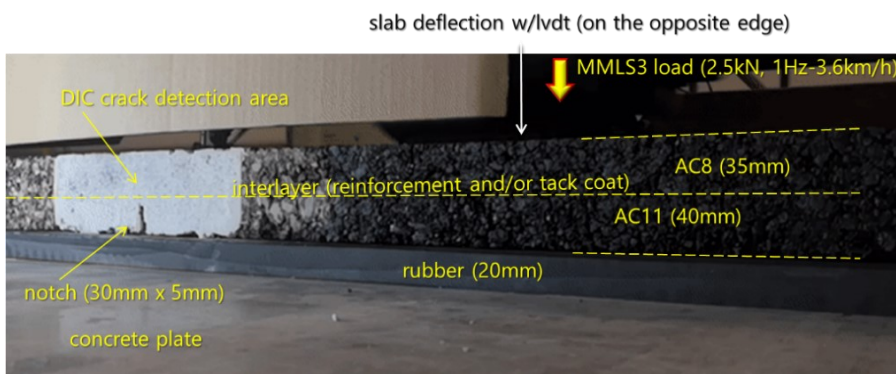


Figure 73: View of the slab under MMLS3 loading

6.2.2.3 Measurements and data analysis

In order to measure how the cracks generate at the tip of the notch and grow through the specimens, different methods were used. As mentioned, each slab was loaded until complete failure, i.e. until the crack reached the surface of the slab. Therefore, it was important to continuously monitor and if possibly quantify the crack progression through the following methods. Figure 74 shows a schema and pictures of the measuring methods, which will be described in detail in the following chapters.

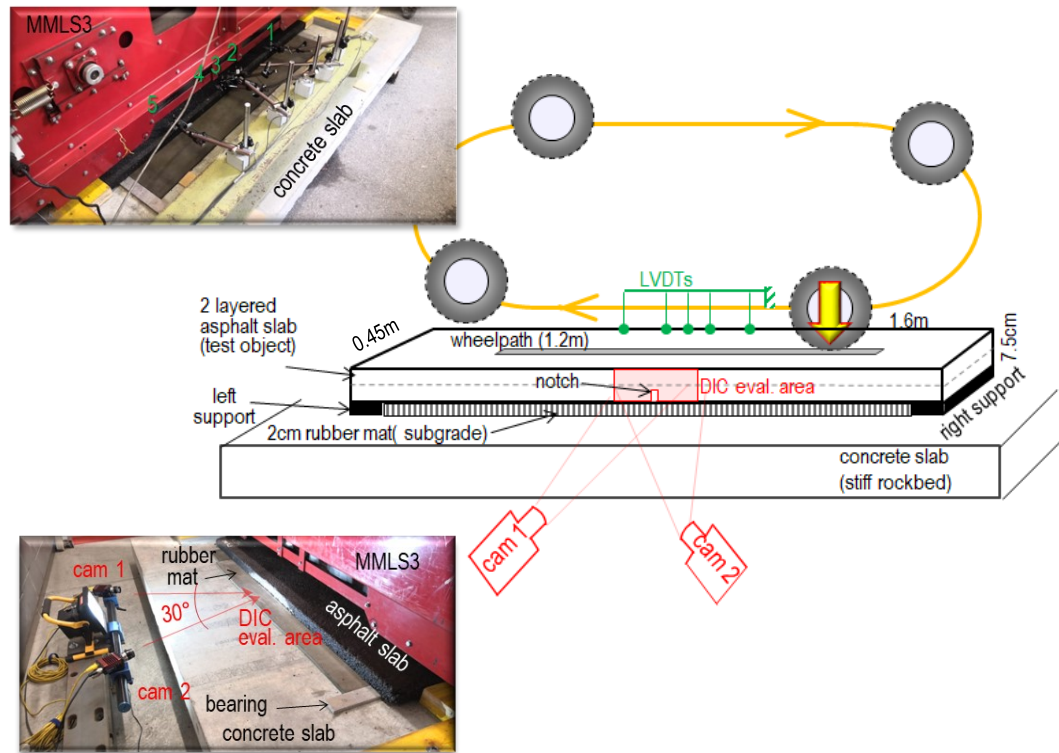


Figure 74: MMLS3 cracking test setup and pictures showing the load application area, DIC System And LVDT measuring arrangement

6.2.2.4 Visual inspections

The simplest technique is a periodical visual observation, taking pictures, videos and making notes of the crack propagation. Therefore, the slabs were regularly controlled to detect cracks on the surface and on both sides. This method for cracking detection is, however, time consuming, inaccurate and the cracks are initially so small that their presence cannot visually be observed but they can only be detected by pressing the tip of the notch with a finger. It is also time demanding and can be carried out during working hours, i.e. during the night there are no inspections. Then, cracks growing overnight can only be detected after several hours.

6.2.2.5 LVDTs

To measure the deflection of the slab during loading, LVDTs were installed close to the edge along the span of the sample. An LVDT was placed in the middle of the span, two LVDTs 50 mm from the center on either side and another two LVDTs 400 mm from the center (see schema and top picture in Figure 74). The deflection was periodically recorded to indirectly determine the crack development. To that end, the deflection measured by the LVDTs is recorded at regular discrete time intervals. In this case, files containing 10 s deflection information of the five LVDTs were stored every 2 min. An example of the time records for the LVDTs located at the center of the slabs is shown in the left part of Figure 75, for two different slabs. Each valley of the curves corresponds to the situation where the tire is in the middle of the wheel path passing close to the LVDT at the center, forming a valley in the time series plot. When the tire reaches the end of the wheel path and the next tire is approaching, the plots form a peak. Once all records are stored, an algorithm calculates the deformation amplitude as the average distance between peaks and valley for each measurement file. Then,

these results are plotted versus the amount of MMLS3 load cycles, as shown in the right side of Figure 75. In this example, it is possible to see that the deflection amplitudes of slab 1 are higher than in slab 2, meaning that slab 2 is stiffer than slab 1. Further, the deflection amplitudes tend to increase with the MMLS3 load cycles, giving a hint of the degree of damage of a slab, as explained below.

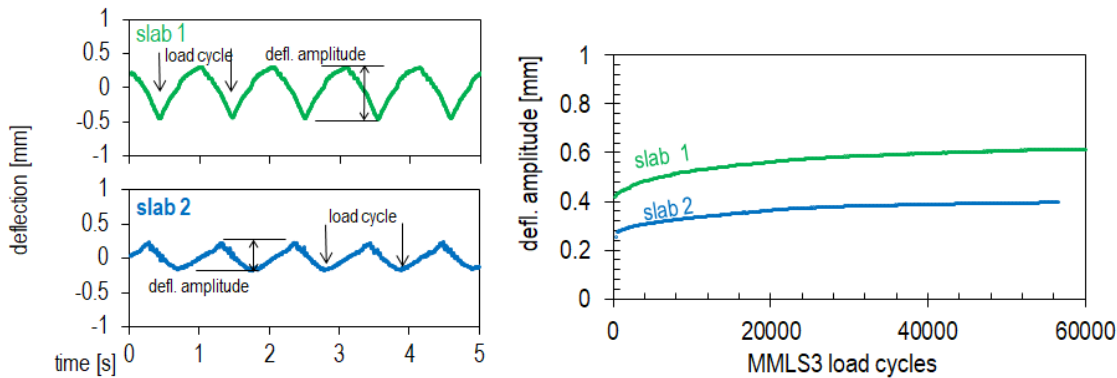


Figure 75: Example of two LVDTs records at the center of two different slabs (left) and calculated defl. amplitudes

As explained earlier, the deflection was periodically recorded to indirectly determine the cracking development according to different phases of the deflections vs. load application curves. In this calculation, deflection (*def*) is defined as the difference between the LVDT reading when a MMLS3 wheel is passing over the sensor (i.e. maximum vertical deformation) and when no wheel is touching the slab (i.e. the slab is not loaded).

Different phases of the deflection curves vs. MMLS3 load cycles are shown in Figure 76, and they represent different damage stages. As previously explained, the deflection amplitudes (*def*, as shown in the left side of Figure 76) are defined as the difference between the LVDT reading when a MMLS3 wheel is passing over the sensor (i.e. maximum vertical deformation) and when no wheel is touching the slab (i.e. the slab is not loaded). Due to the damage the deflection amplitude will increase with the accumulation of MMLS3 load applications. A certain initial value def_0 will increase to a final value def_{END} when a fatigue crack progresses from the bottom to the top of the slab. Ideally, the process can be divided into three phases.

- Phase I is marked by a rapid rise in deflection amplitude, caused by the settlement of the system under load.
- Phase II begins with the propagation of a micro-crack from the notch tip, extending until it reaches the interlayer plane. This phase is characterized by a relatively low, linear, but steady increase in deflection amplitude. During the tests, this phase was also observed to correspond with the development of debonding (i.e., horizontal crack propagation) and increasing shear creep deformations between the layers.
- Phase III is characterized by a sudden, sharp increase in deflection amplitude, signaling the formation of a macro-crack. This crack, which began as a micro-crack at the notch during Phase II, progresses vertically through the top layer of the slab, eventually causing complete failure and separating the slab into two parts, thereby marking the end of the test.

It should be noted that, depending on the grid reinforcement system, this idealized three-phase damage mechanism may differ, with certain phases potentially not being clearly distinguishable.

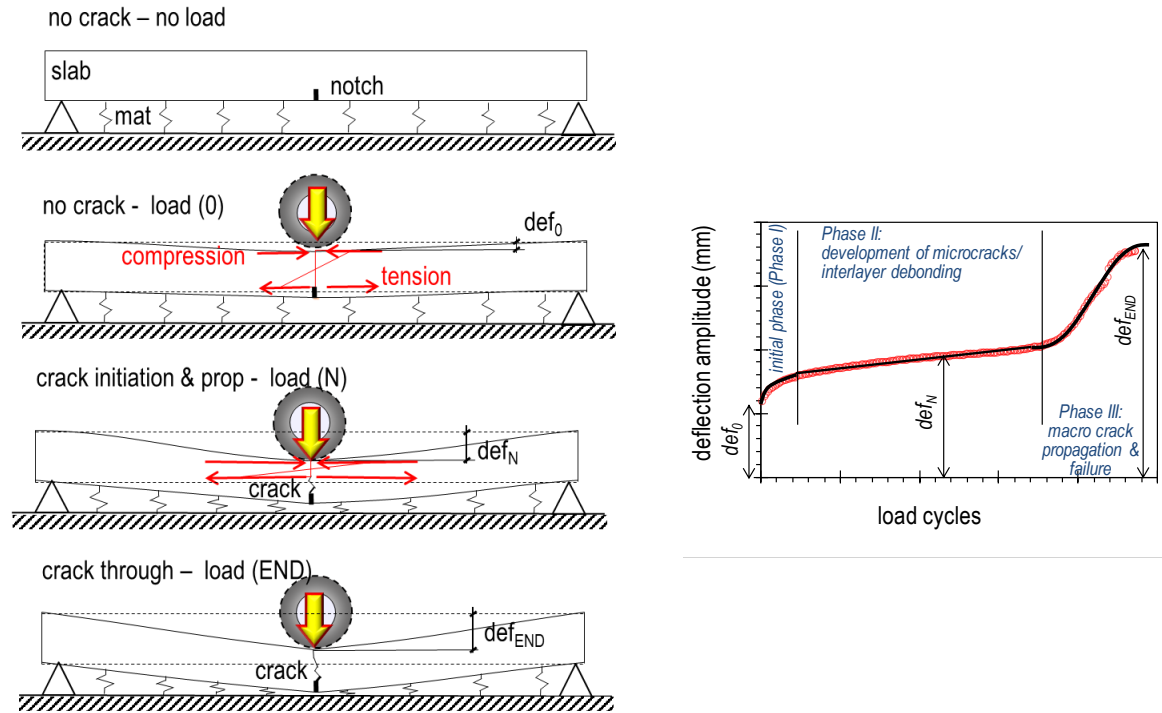


Figure 76: Conceptual schema of the MMLS3 test progression (left) and expected evolution of deflection (right)

Using this method, the approximate number of MMLS3 loads required to damage the slab can be inferred. However, a more precise and straightforward method was used in parallel to the LVDTs, as explained in next section.

Digital Image Correlation (DIC)

In addition to the measurement of deflection, the crack development was monitored by digital image correlation (DIC). As explained in the previous chapter, this non-contact optical technique measures the deformation of a body under load by tracking and connecting the displacements of random speckle patterns applied to its surface. Since the experimental and the cameras used for these tests are different than the ones for the cyclic loading on beam specimen tests, a detailed description of the measurement and data analysis process is given below.

The lateral surface around the notch, about 15 cm in each direction, was grinded to make it smooth and was painted with white color to provide high contrast. A black dotted pattern was sprayed with a high-pressure nozzle to create the DIC speckles (see Figure 77). Two cameras placed at an angle of about 30° and pointing at the notch monitored this area (see bottom picture in Figure 75). Image recording was triggered periodically, every ca. 1200 MMLS3 load cycles. Each image recording comprised 50 frames taken in a 2 s window, i.e. enough to capture the passing of the MMLS3 wheels with a sampling rate of 25 Hz as shown in Figure 78. This figure displays five different situations, where one of the tires starts to touch the slab surface, then rolls towards the center of the slab and reaches it after 0.5s. At this time, the bending of the slab is

maximum, and a potential crack opens. Then, the tier continues its travel toward the end of the wheel path and after 1 s the slab is again and for an instant not loaded and the bending is minimum, closing any crack. The deflection of the slab center is also presented at the bottom graph of Figure 78.

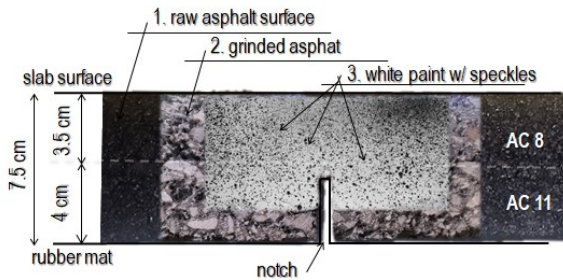


Figure 77: view of the measuring area showing the raw and grinded surface and the surface after painting and speckle application

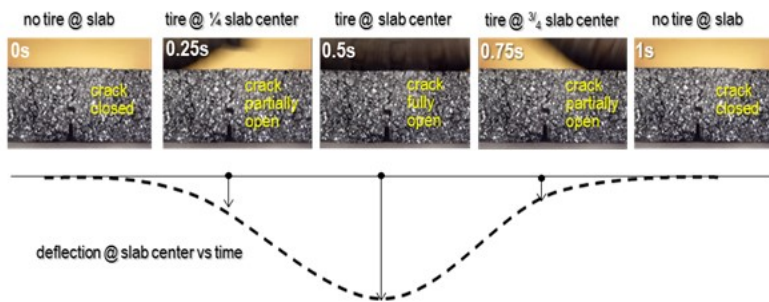


Figure 78: Schematic of the tire passing over the notch, showing the opening of an existing crack whereas the bottom graph shows the deflection of the center at different times

The software used for image acquisition and post-processing was the Istra4D (v. 4.4.2). In order to detect a crack, images of the non-loaded state (when no wheel is touching the slab, see 0 s and 1 s frames in Figure 78) were compared with the loaded state (see 0.5 s frame in Figure 78). The deformation of the surface is obtained by recognizing a pattern- speckle followed throughout time (Figure 79). By doing this, it is possible to evaluate displacement and deformation present in the pattern studied. To be able to recognize a dot pattern, the image captured by the DIC cameras was converted into binary data. In this process each pixel color is converted into a number. Once the image is transcript in memory through numerical values, it is possible for the DIC software to compare it to the next image after a time interval (Figure 43). In this new image, the program will try to find the pattern identified in this first image using a correlation function. By doing so, it will be possible to know if the pattern moved or not, thus knowing if deformations or rigid body displacements occurred. To find the subset pattern in the new image, a correlation function is used. By testing multiple values in this function, different possible locations of the pattern are tested until the function determines which pattern location has the smallest error when compared to the original one. This location is thus saved as the $n+1$ location. When comparing the n and $n+1$ locations, it is possible to determine the displacement of the pattern matching the original pattern with a relative displacement. Perfect matches rarely occur due to

distortions in the material and image noise slightly modifying the colors from one image to another. In most cases, the selected location is the one that gives the smallest error through the correlation function. As a result, the relative displacements can be displayed over the image of the observed element though a color code. Moreover, starting from the relative displacements the program offers visualization also in terms of strains. In Figure 80, the von Mises strains obtained by comparing the non-loaded state and the loaded state for different number of MMLS3 loads is presented, clearly showing the progression of the cracks. It can also be observed that the cracks tend to grow wider and the strains in this area tend to increase. This is due to the progression of other cracks in the slab and due to the fact, that because of the relative movement of the crack walls, any interlocking between the aggregates on both sides of the crack tend to grind reducing the stiffness of the slab.

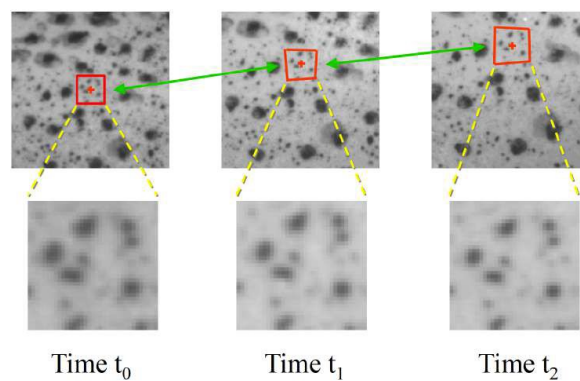


Figure 79: DIC pattern following over time (from Digital Image Correlation: Theory [PDF]. (Sutton et al., 2009))

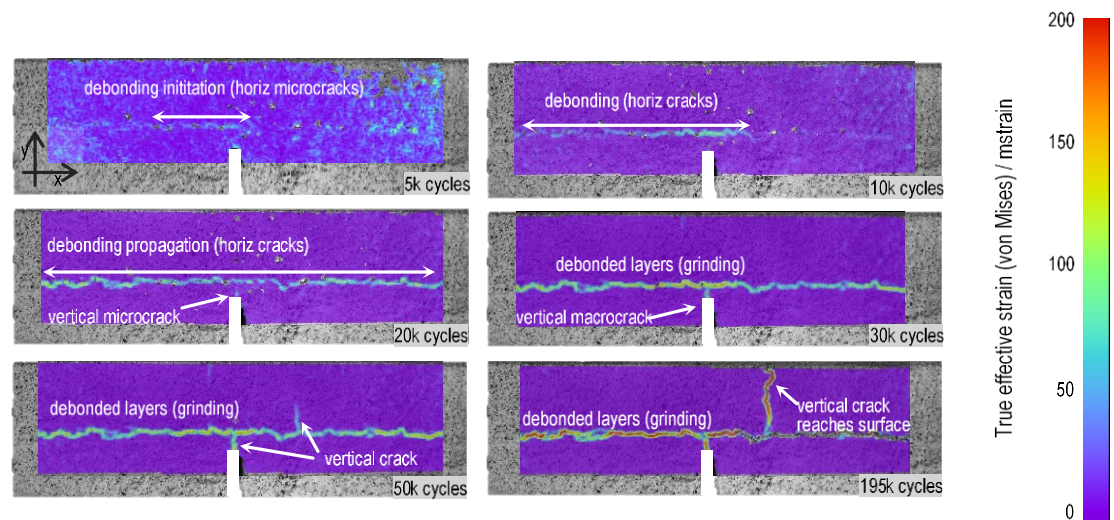


Figure 80: Example of the evolution of the von Mises strains showing the evolution of the cracks vs. the MMLS3 loading; each figure

In order to compare the susceptibility of the different reinforcement grid to the crack formation and propagation, an algorithm to quantify their length was designed and programmed in Matlab. Firstly, all the results from the DIC software Istra4D were exported as ".hdf5" files. These files have all the information about the measurements in a format that can be processed by Matlab. The algorithm searches for all .hdf5 files

in a folder and sorts the stain information for each measurement frame and stores it as an array, where the dimensions are equivalent to a pixel, and three values correspond to the color in the RGB scale (see example in Figure 81, part a)). Then, the colors are converted to grayscale values, with values ranging from 0 to 1. The frame when the tire is passing over the center of the slab is selected by taking the lowest y displacement value of a random point and only this frame extracted for the crack analysis. A threshold is defined (0.1 worked for most of the analysis) to define the strain induced where a crack is present, and in those cells where the threshold is exceeded, the grayscale values are replaced by a 1. If the cell value is below 0.1, the value is replaced by a 0. Consequently, the algorithm returns a matrix (area covered by the DIC cameras) with 0 and 1 that when plotted, forms a black and white image showing the crack position. Afterwards, the image is skeletonized, i.e. the crack width is represented by only one pixel, and it doesn't depend on how much the cracks open under load, i.e. it results in a thin version of the crack that is equidistant to its boundaries. Then, the length of the cracks is calculated in the horizontal and vertical direction as the sum of all the pixels in each direction. Finally, a percentage is calculated by dividing them though on the image width and high in pixels, respectively. The analysis is repeated for all the measurements and the number of MMLS3 cycles is calculated based on the timestamp of the measurement, retrieved from the ".hdf5" file. An example of the image processing is presented in part a) of Figure 81, for 196 k MMLS3 load cycles. In the chart shown in Figure 81, b), the crack progression in the horizontal and vertical direction is shown. The test is considered as finished when the vertical crack covers 100 % of the pixels between the tip of the notch and the top of the image.

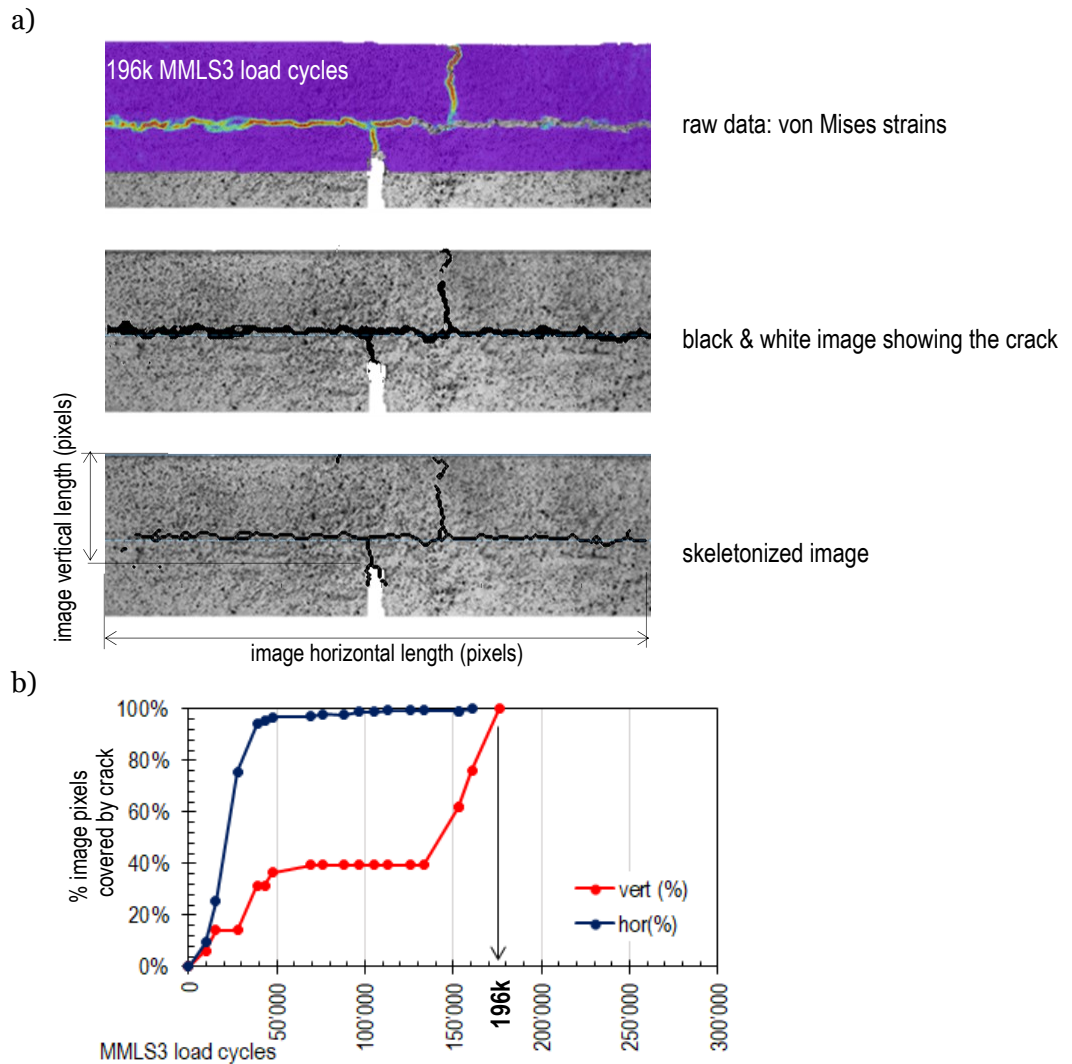


Figure 81: Example of the image process to quantify the cracking progress through the horizontal and vertical crack length for a) 196k number of MMLS3 load applications and b) example of a chart with the evolution of the crack length

Temperature

The temperature of the slab during a loading was measured in the slabs and the values were recorded with their timestamps every 2 min. To that end, a thermocouple was installed by drilling a 10 cm hole on one side of the slabs, between the two layers (see Figure 82). The values were used to keep the slab temperature between 21 °C and 19 °C by manually controlling the target temperature in the chamber.



Figure 82: View of the thermocouples and data acquisition system

6.2.2.6 Test plan

In the first stage, 3 trials were carried out in specimens called KP1 to KP3. As already explained in § 6.2.2.2, these tests required long testing times (more than 3 weeks) and the expected crack progress to the slab's surface was not taking place (see next section). With the reduction of the slab geometry and load increase, the tests were able to be finished within a working week. Using this setup, a total of 3 specimens per system (systems A, B, C, D and unreinforced system) were produced, totalizing 15 slabs. The unreinforced specimens were treated with a tack coat in a concentration of 200 gr/m² of the residual bitumen. The color code for the presentation of the results is as in the rest of the report: for the reference is black, for System A is yellow, for System B is blue, for System C is red and for System D is green. Table 9 summarizes the names of the specimen tested and presents a Gantt chart with the testing sequence.

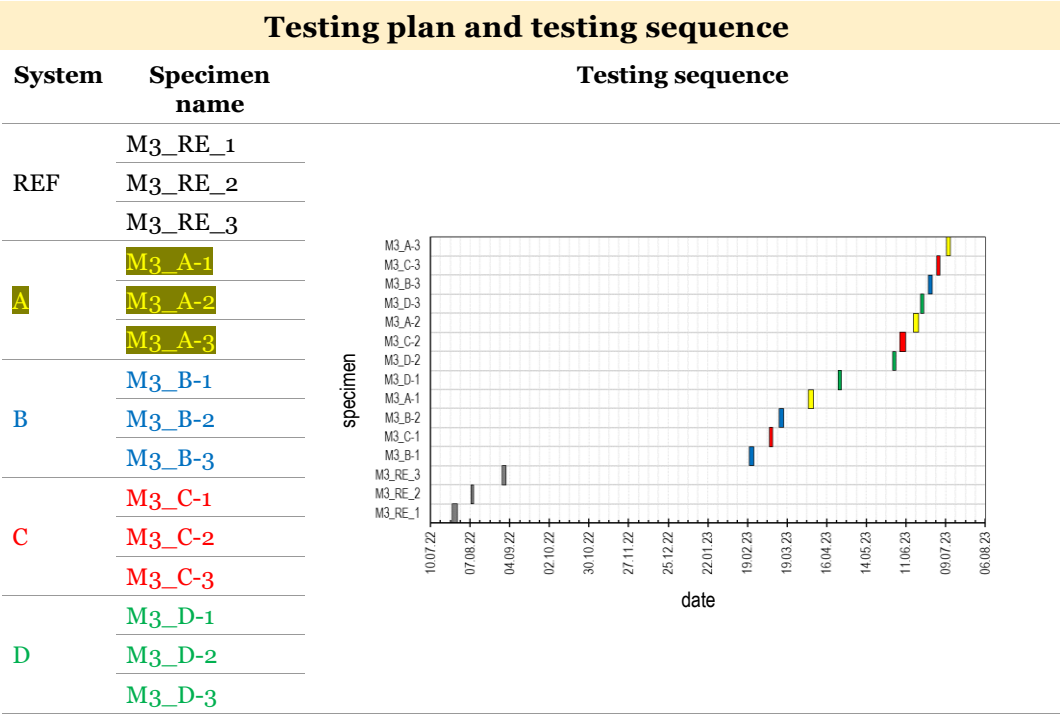


Table 9: Name of the systems, specimens and chart of the testing sequence

6.2.2.7 Results

Figure 83 shows one of the results for the trial tests in KP1 to KP3 in terms of deflection amplitude and cracks with DIC. Although an extensive testing effort, no crack reached the slab surface forcing to a redesign of the test setup.

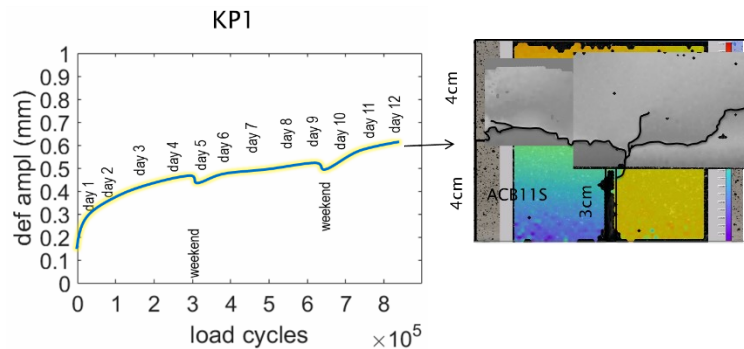
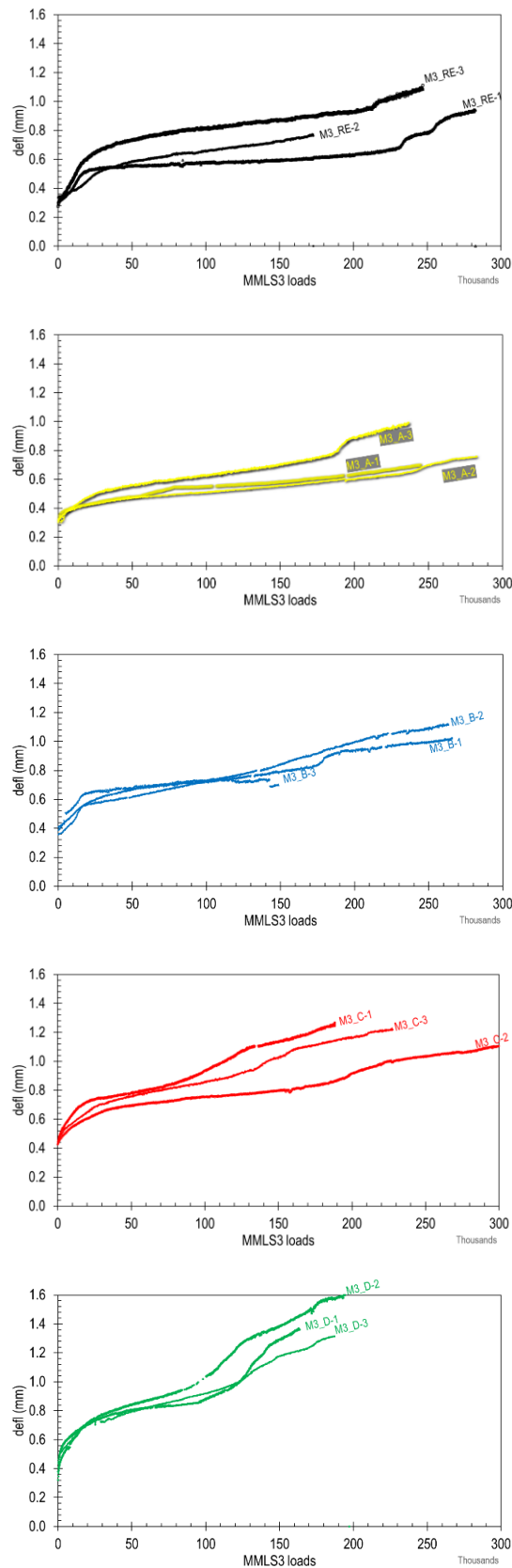


Figure 83: Results for one of the trial 3 slabs tested under different conditions than the rest of the 15 specimens

In Figure 84, the deflection amplitudes (LVDT located in the center of the slab) vs. MMLS3 load cycles for all the 15 slabs tested are presented, showing a relatively good repeatability. Note that for every reinforcement System As well as for the reference system 3 individual specimens were tested. Most of the time the deflection amplitude curves in Figure 84 show three different stages. In an initial loading stage (a few thousand cycles), they grow rapidly which can be attributed to the formation of microcracks. While the second stage is characterized by a steady state linear deflection amplitude increase. In this stage, micro cracks grow though the specimen in different ways, as explained in more detail later in this chapter. In the third and final stage, the crack reaches the top of the specimen causing a separation in the specimens producing an exponential increase in the deflections. In order to be able to compare them to each other, in figure Figure 85 the average of the 3 slabs for each System Are plotted together. It is possible to observe that the different systems studied have different deflection values, indicating that the reinforcement placed between the slab layers provides different stiffnesses to the system: if the material and geometry of the slabs layers are the same, the load and speed are the same and the temperature is the same for all slabs, the stiffness only depends on the grid and its interaction with the material in which they are embedded. It can be observed that the deflection of System A is the smallest, whereas the deflection for System D is the highest. This means that System A is the stiffest and System D is the most flexible one. This undoubtedly implies that the tensile stresses generated at the notch tip as the simulator wheel passes will be greater in the more flexible systems, which will accelerate fatigue and decrease their service life.



REF

A

B

C

D

Figure 84: Displacement amplitudes vs. number of MMLS3 loads measured by the lvdt 3 (center) for the 15 tested slabs

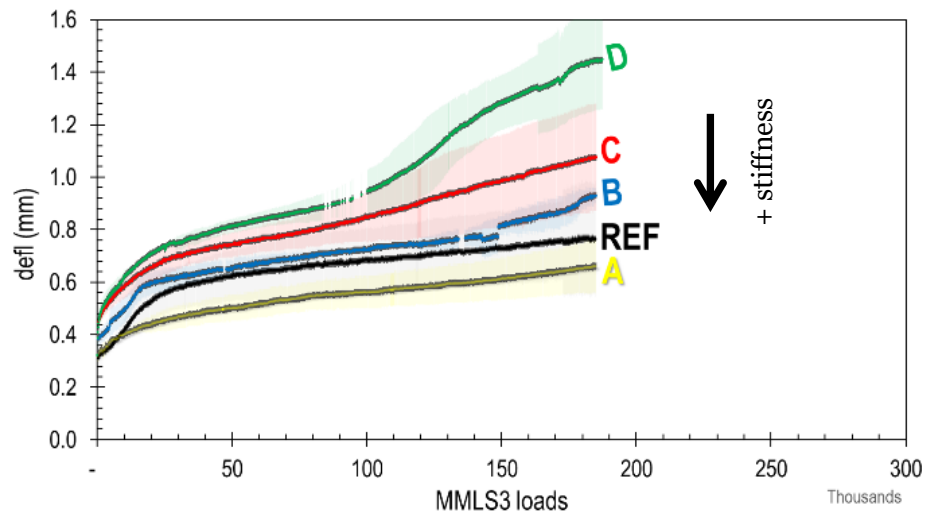


Figure 85: For each System Average deflection amplitude vs. no. of MMLS3 loads

The DIC evaluation results, as described in 6.2.2.3, are presented in Figure 86 until Figure 90 for each of the tested slabs. The crack shape at the end of the tests, as processed by the Matlab algorithm is shown on the left side of the figures, and the calculation of the horizontal and vertical cracks progression are on the right side.

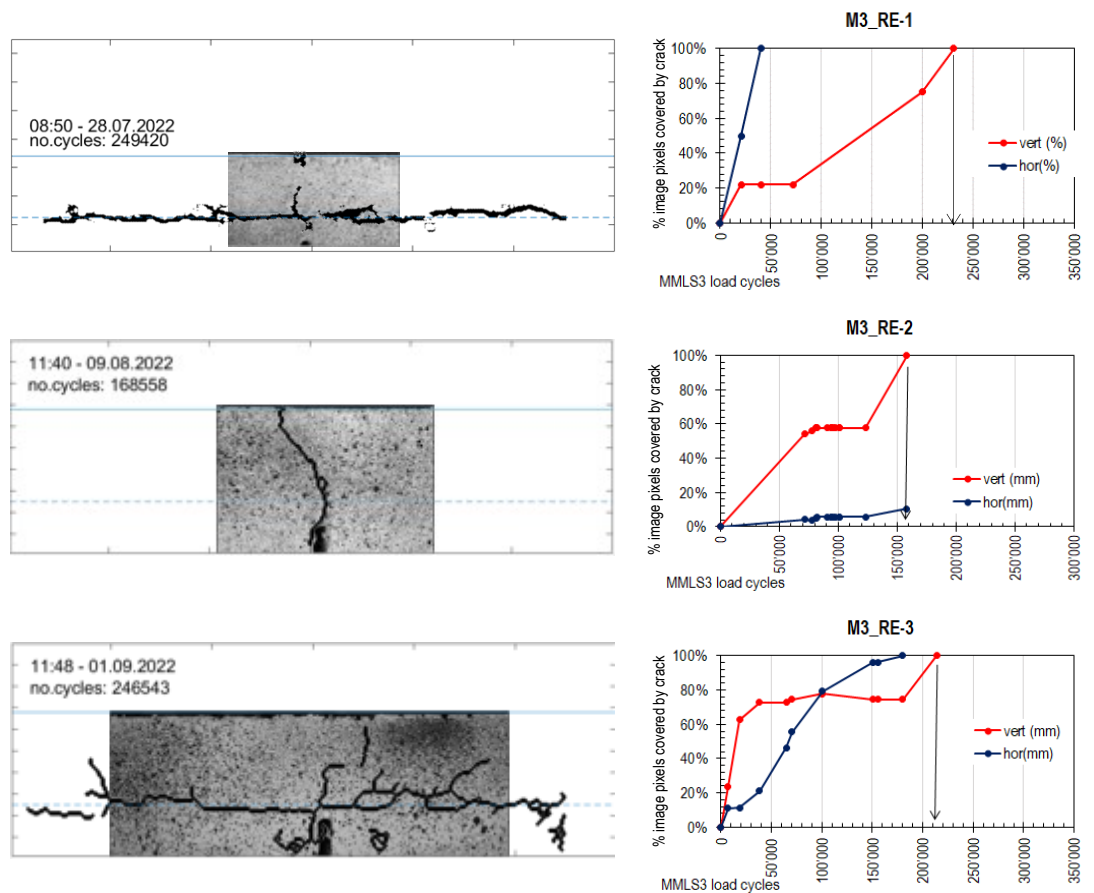


Figure 86: View of the crack pattern obtained with the Matlab algorithm (left) and calculated horizontal and vertical crack length (right) for the reference slabs

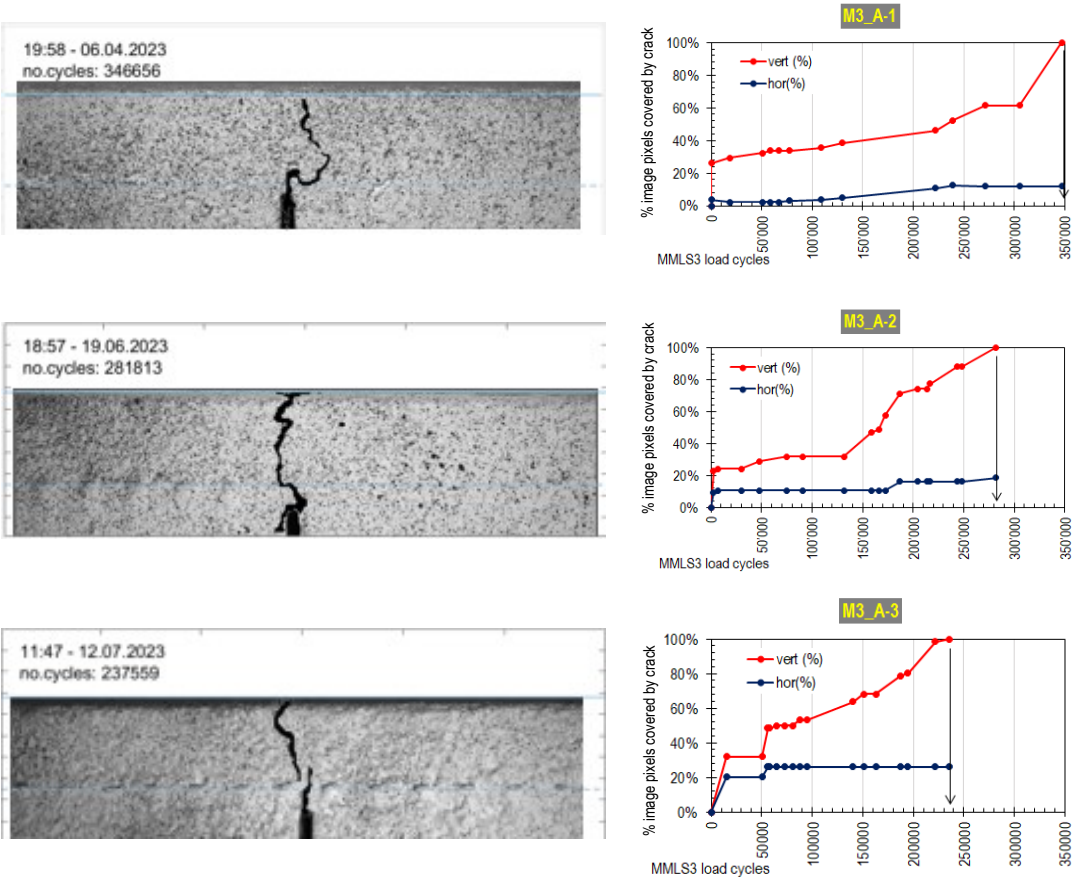


Figure 87: View of the crack pattern obtained with the Matlab algorithm (left) and calculated horizontal and vertical crack length (right) for System A slabs

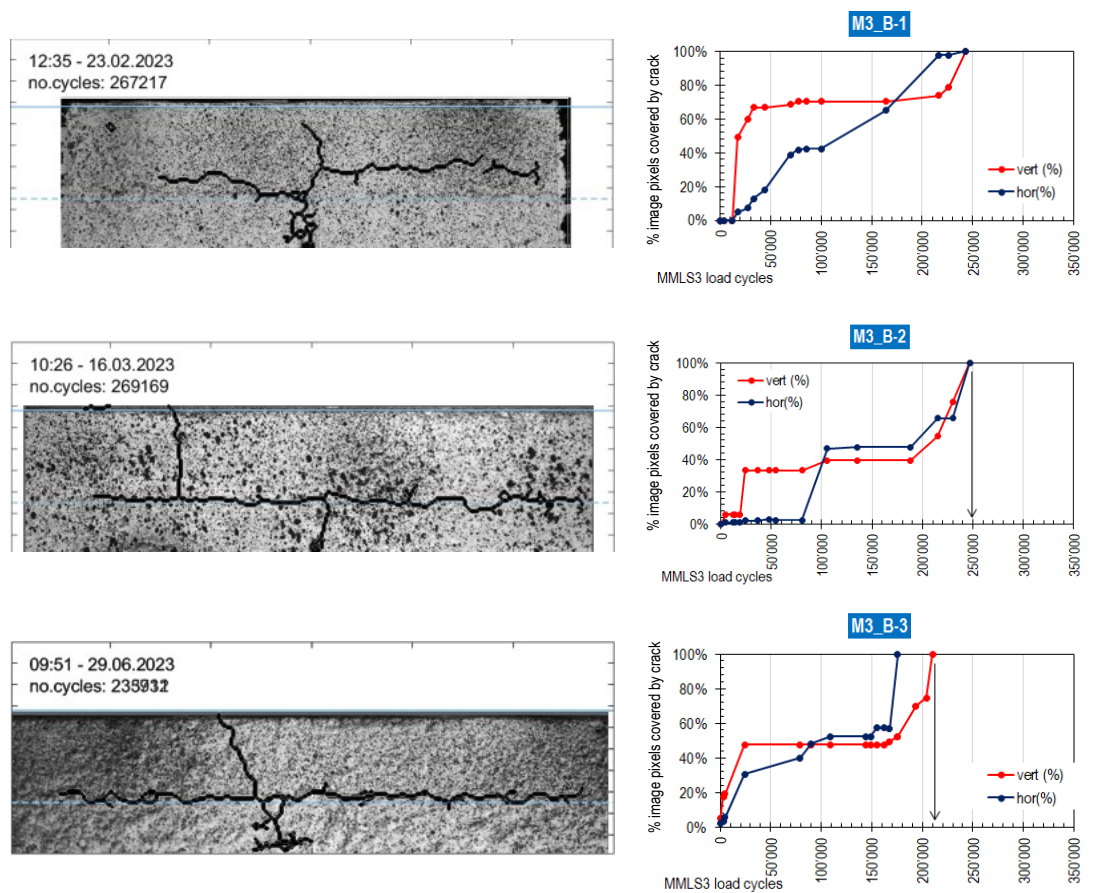


Figure 88: View of the crack pattern obtained with the Matlab algorithm (left) and calculated horizontal and vertical crack length (right) for System B slabs

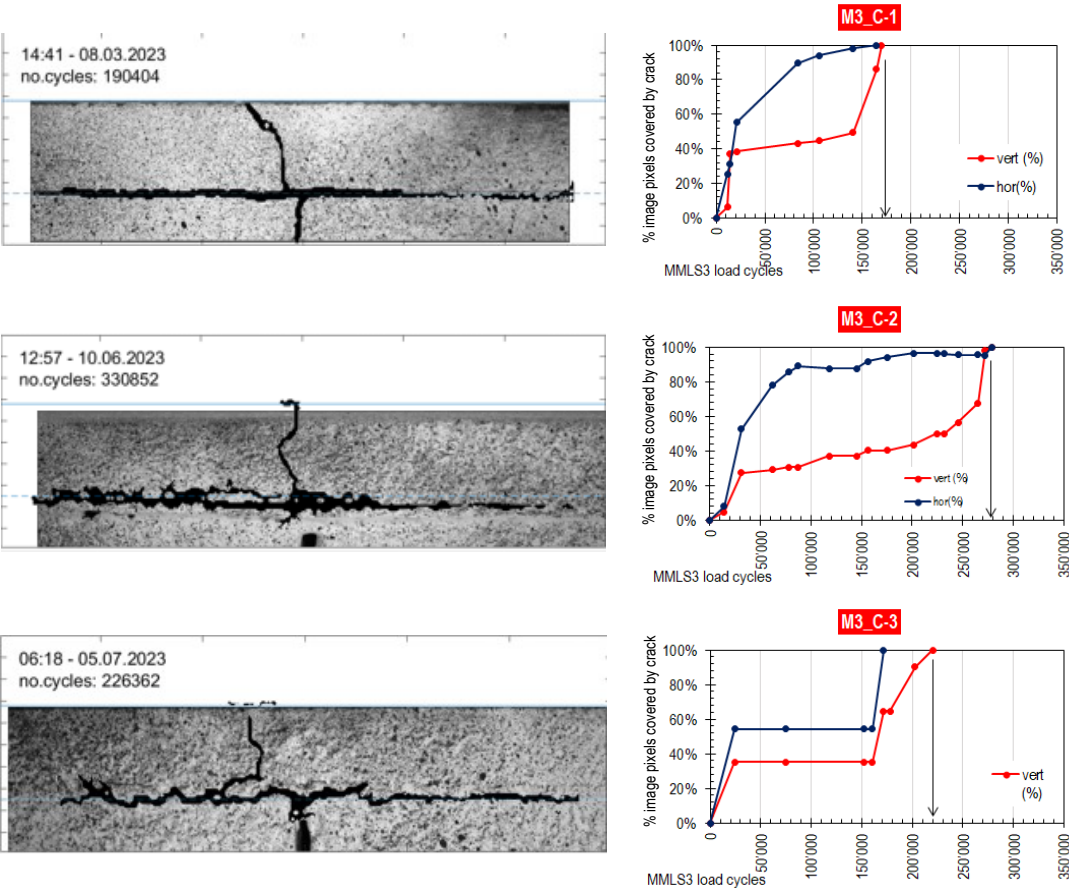


Figure 89: View of the crack pattern obtained with the Matlab algorithm (left) and calculated horizontal and vertical crack length (right) for System C slabs

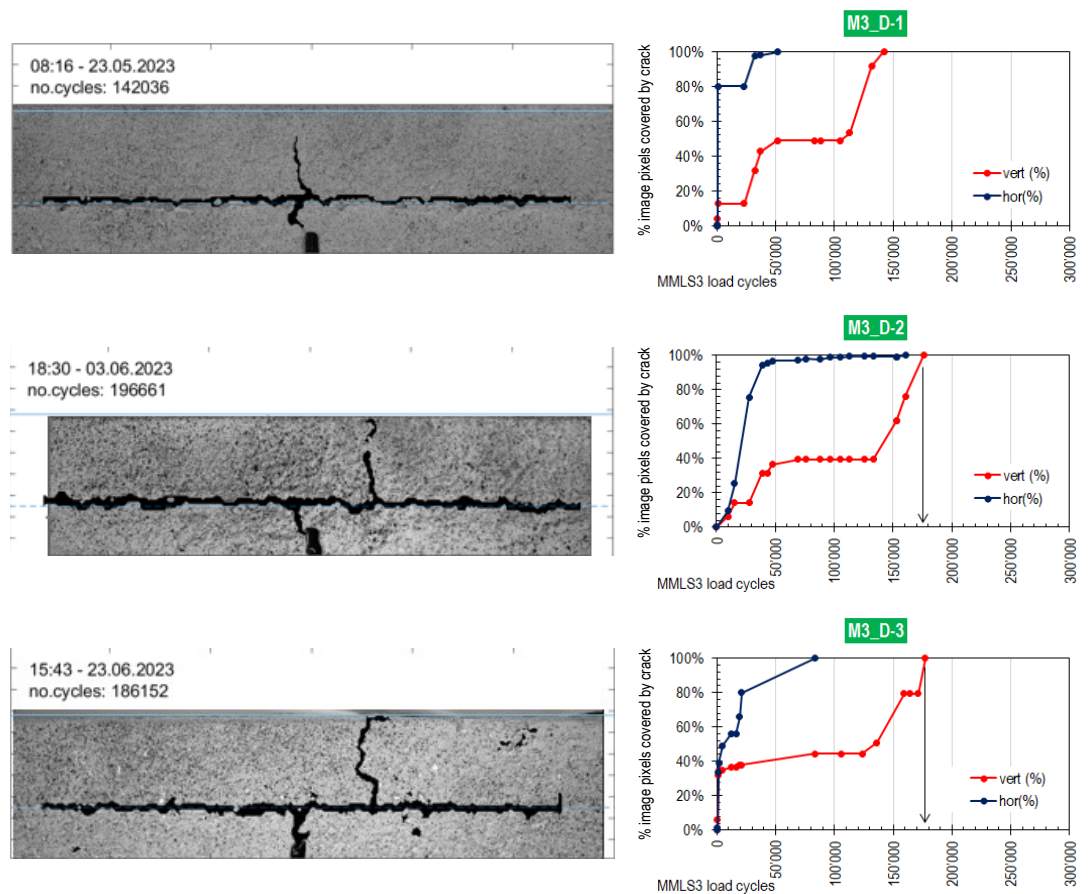


Figure 90: View of the crack pattern obtained with the Matlab algorithm (left) and calculated horizontal and vertical crack length (right) for System D slabs

Through the analysis of the images, basically three different mechanisms of crack formation and propagation could be identified. For some specimens, immediately after the initiation of the loading, the DIC analysis shows a debonding of the layers, followed by the initiation of a crack at the tip of the notch which travels through the slab thickness to the upper surface. In the second damage process, a vertical crack starts at the notch propagating directly to the interface and leading to a debonding of the layers, before travelling further to the top. This means that the interlayer is slightly stronger than in the previous case. For both situations, the vertical crack on the top AC 8 S layer can be a continuation of the crack in the bottom AC B 11 S layer or it can be displaced horizontally. The lateral dislocation of the vertical crack is a sign that the interlayer is extremely weak and both layer work independently under load. The third and last damage mechanism identified is when a vertical crack directly propagates to the top layer without causing a separation at the interface. This classification of three crack formation and propagation process is summarized through Figure 91.

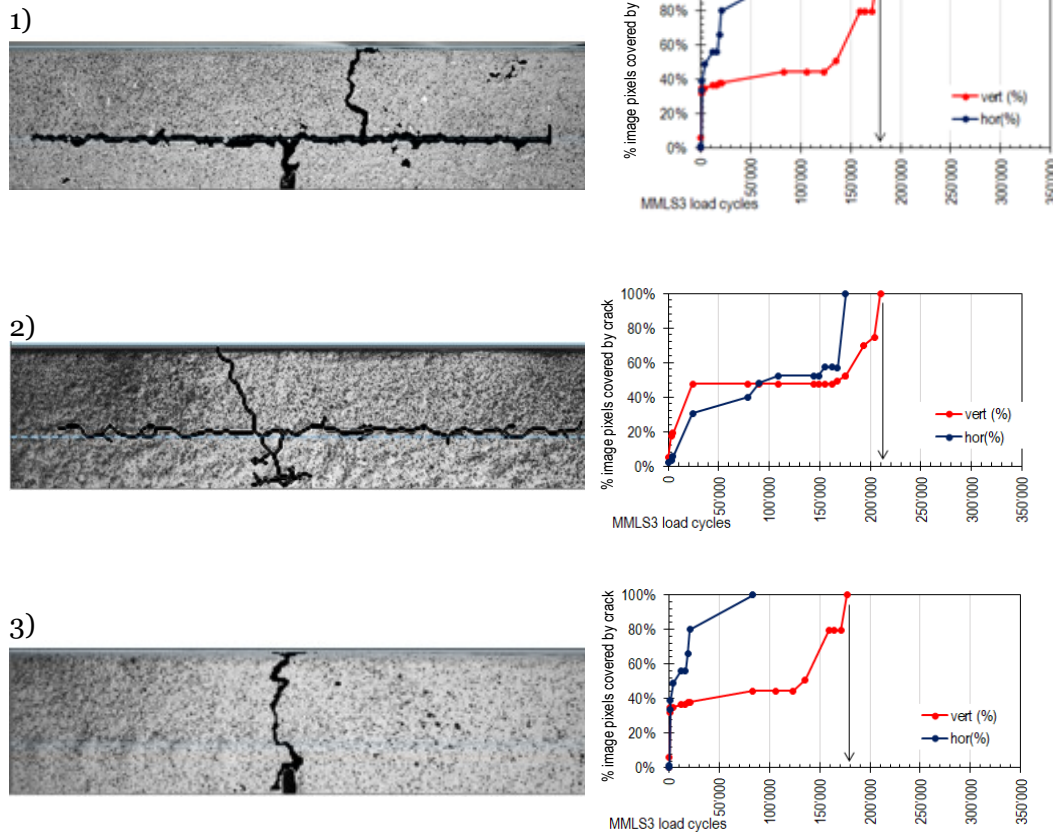


Figure 91: Three mechanisms of crack formation and propagation identified through the DIC analysis

In the first two cases, it was observed that crack propagation occurred generally faster than in the slabs where no separation at the interface between layers was observed. The failure type can be generally correlated to the performance of the slab, i.e. if the slab presents interlayer debonding, the number of load cycles until failure tends to be lower than if the interlayer does not debond.

Figure 92 presents the ranking of results averaging the 3 slabs for each grid System And the reference, normalized to the system with the best performance (highest number of load cycles until failure). According to this chart, the grid system with the best performance is A, while D performs poorly, only able to carry ca. 60% of the loads of grid A. Nevertheless, all reinforcement systems except System D outperform the unreinforced reference.

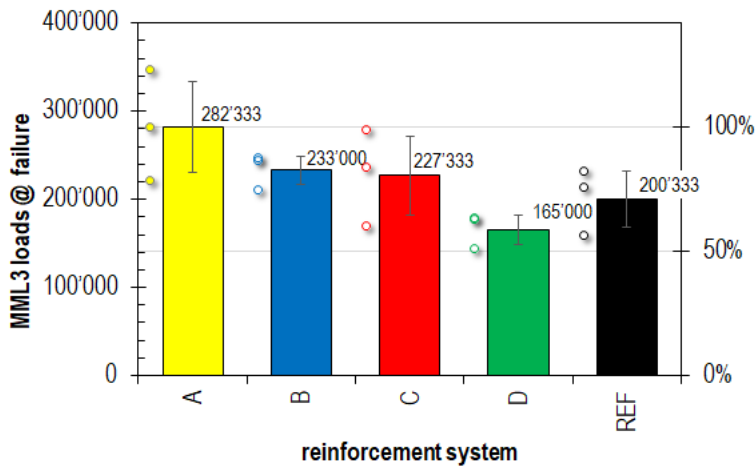


Figure 92: Approximate number of loads until failure of the slabs

7 Modelling

Although the literature review as well as the laboratory results presented in this work show that, in general, reinforcement grids have a positive effect in a pavement performance extending its life, their use is not widely accepted. One possible explanation is the fact that there is a lack of methods for structural dimensioning and their use and application rely mostly on experience. To some extent there is limited modelling using analytical or numerical methods available, but these methods are not able to reveal their potential benefits. This constitutes a major obstacle to the acceptance and integration of reinforcement systems in road construction. In addition, it is not clear how the different commercial products interact with the pavement under certain conditions, or which intervention would be the best under specific circumstances. Questions about how a particular reinforcement type works under load in combination with a specific temperature and a certain pavement structure can only be answered through a simulation of the conditions. Accurate modelling of a pavement reinforced with a grid is therefore essential for the correct design and dimensioning of the pavement structure. Further, estimating the theoretical service life extension of a reinforced pavement is possible only by obtaining the stress and strain conditions using models that precisely simulate the response of the pavements to loads. Several modelling methods used in the design of pavements, mainly in the mechanistic empirical dimensioning methods, are of analytical nature and often use 2D multi-layered geometries. However, these methods require assumptions that sometimes include oversimplifications to obtain the stresses and strains required for proper dimensioning. In the case of the installation of reinforcement grids, the complex structure and the interactions between the reinforcement and the rest of the pavement makes it more difficult to obtain trustful results when using the abovementioned methods.

Therefore, the objective of this section is to establish the basis for taking reinforcement grids into account in the dimensioning process. In order to do this, the finite element method (FEM) was chosen as the most suitable tool for modelling the complex geometry of the grids and the interaction between them and the asphalt pavement. The finite element model used in this project was based on the commercial, Abaqus code. In a first stage, a model of the relatively simple Leutner test was prepared using Abaqus/CAE environment to establish the most appropriate interlayer bonding model and its parameters. CAE is an intuitive interface that allows to prepare models using a user-friendly windows design process, suitable for these relatively simple simulations. For the simulation of the more complex MMLS3 loading test, the CEA module was not used directly. Instead, in this case a plug-in was created the Python API following the structure from a previous project. The plug-in consists of an interface accessible through the Abaqus/CAE environment where the user can enter geometry, temperature, loads, speed, and other data to simulate and evaluate the effect of a passing MMLS3 tire on top of a slab. It also comprises a material and load library that allows to study the effect of different asphalt mixtures or different reinforcement grid materials. As a result, whether using CAE or using the GUI plugin, the software creates an input file (model_name.inp), where all the information about geometry, properties

and test conditions can be read and executed by an Abaqus solver. As in the case of the Leutner test the load is applied at a low speed, the Abaqus Standard solver was used. On the other hand, for the MMLS3 simulations the solver used was Abaqus Explicit, as the tires roll at relatively high speeds and the dynamics forces play an important role in the system response. The following chapters describe the model characteristics and presents some simulation results.

7.1 Modelling the Leutner test

7.1.1 Overview and tests used for the calibration of the model

The model presented in this section had the goal to simulate the test used for the estimation of the interlayer bonding quality, with the aim of using the results for modelling the layers interaction in the MMLS3 simulations described in next section. Therefore, the most important parameter to simulate is how the interlayer is behaving, how is the bonding strength when a pure shear load is applied. Physically, the interlayer is a very thin layer with various mechanisms reacting against the shear forces, thanks to the interlocking between aggregates of both AC layers and the adhesion provided by the tack coat. To evaluate and calibrate the models developed under different conditions, Leutner tests were carried at three temperatures without tack coat and for one temperature with tack coat. The average results of 5 repetitions of specimens with and without tack coat at temperatures of -10°C , 20°C and 40°C (only 20°C for the tack coat case) are presented in Figure 93.

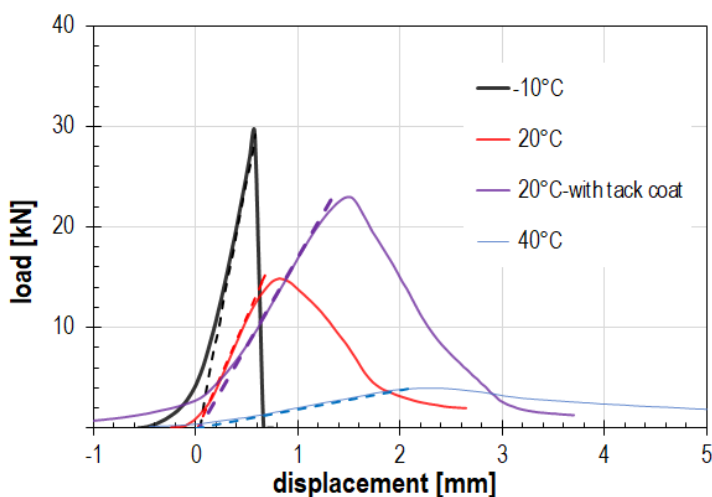


Figure 93: Test results used for the calibration of the Leutner model

7.1.2 Units

Abaqus does not have a built-in system of units, consequently a consistent set of units has to be defined and all the values physical values of the model have to be consistent. The selected unit system for all the simulations is shown in Table 10.

Coherent units

Quantity	SI (mm)
Mass	ton
Length	mm
Time	s
Force	N
Stress	MPa
Energy	N-mm
Temperature	°C
Gravity	9.81e+3

Table 10: Units used for the Abaqus models

7.1.3 Geometry

In terms of geometry, the model was considered to be a cylinder with a length of 100 mm and with a diameter of 150 mm. It was modelled with one deformable solid 3D part. It was then divided into three sections along its length: 0 mm to 29.9 mm presenting the top asphalt layer, 29.9 mm to 30 mm representing the interface and 30 mm to 100 mm the bottom asphalt layer, as shown in Figure 50. For each of these layers a different material was assigned, respectively asphalt concrete AC 8 S, the interface and asphalt concrete AC B 11 S.

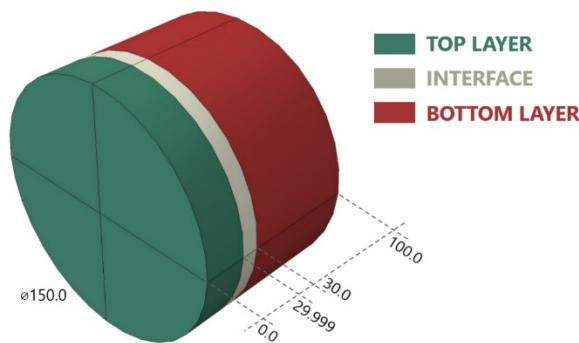


Figure 94: Leutner model geometry in Abaqus (values in mm, not scaled)

7.1.4 Materials

The asphalt concrete layers of the specimen are defined in the numerical model using the results of the laboratory tests shown § 5.1, as described in next section. The modelling of the interlayer bonding, which is actually more relevant than the layers' materials, is explained in detail in § 7.1.4.2.

7.1.4.1 Layer material AC 8 S and AC B 11 S

The two asphalt layers are modelled assuming a temperature dependent isotropic elastic behaviour with a Poisson ratio of 0.3. For each temperature, it is assumed that the Young's modulus value used is the one from the Indirect Tensile Test results, at 10Hz. In fact, because the test is quasi-static, the value from the highest load frequency is used. The different values are shown in Table 11.

Material properties for Leutner test modelling

Poisson's Ratio	Temperature [°C]	Young's Moduli [MPa]	
		AC 8 S	AC 11 S
0.30	-10	23847	24903
	0	20515	20881
	10	13554	14675
	20	9397	9939
	40	1083 *	467 *

Table 11: Young Moduli values for AC8 and AC11 (*linear extrapolation from data)

7.1.4.2 Tack coat (interlayer)

Two different methods gave promising results for modelling the interlayer. The first is the use of an Extended Finite Element Method (XFEM). Attempts to use this method have encountered many problems, specifically it cannot be perfectly adapted to the geometry of the test. The idea behind the method is to define elements for the mesh that can be split in two to simulate the propagation of a crack. The implementation of this method in the various numerical models is described in detail in the Abaqus software user guide. The main issue when using XFEM is that only the elements at the interface are allowed to propagate the crack. Once an element has cracked, the position of the crack is kept. Thus, if the crack leaves the plane of the interface, as shown in white in Figure 95, for the software it is not possible to separate the two layers completely. This "zig-zag" crack propagation pattern is systematically obtained when using XFEM, even when the mesh is refined, such as done here with two elements for the interface thickness. This leads to excessive deformations, and the result obviously does not correspond to reality. No modelling attempt was able to overcome the problem, so the model was not used any further.

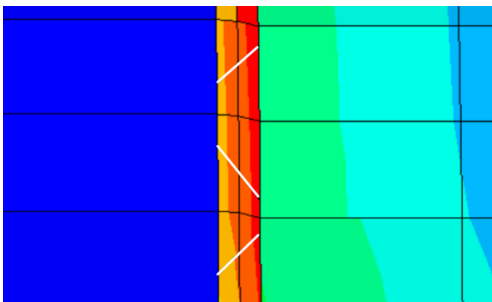


Figure 95: Crack propagation using the XFEM

The second method is the use of cohesive elements in a Cohesive Zone Model (CZM). This modelling method is more suitable for a shear test and examples of its use can be found in the literature. The model can be used to define the post-peak softening behaviour of concrete but also gives a suitable behaviour for HMA material. The idea is to build up the interface with cohesive elements with a thickness close to zero. For this a number of steps is necessary:

- Choose the right element type
- Include the elements in the model geometry and connect them with each other
- Define the initial geometry of the elements

- Define the mechanical properties of the elements

Abaqus allows the use of several types of cohesive elements, but the COH3D8 elements, i.e. a three-dimensional parallelepiped element with a node at each vertex, were the most suitable for this model. Several rules must be followed for modelling with cohesive elements. These elements can only be used in the thickness of a single element, linking two components together, i.e. the interface between the two layers will only have one element thickness. It is important that the cohesive elements created share the same nodes as the elements to which they are connected.

It is possible to manually specify the initial thickness of the cohesive elements. However, the most accurate results were obtained with a default initial thickness of 1. This is an assumption made by Abaqus, which considers that it is always more appropriate to use a traction separation law only with elements with a thickness close to zero. Thus, this default choice ensures that relative separation displacements are equal to the nominal strains. Cohesive elements are associated with an orientation that must be identical for all elements of the same interface, i.e. the same top and bottom surfaces. This means checking that all the elements in the interface share the same stack orientation.

Modelling with cohesive elements requires the choice and definition of a constitutive response. When modelling a thin interface of almost negligible thickness, it is preferable to use a traction separation law. Subsequently, changing the parameters of this law is the main tool used to calibrate the model. The law assumes that the initial behaviour of the elements is linear elastic, followed by the initiation and evolution of damage. A typical traction-separation response is shown in Figure 96

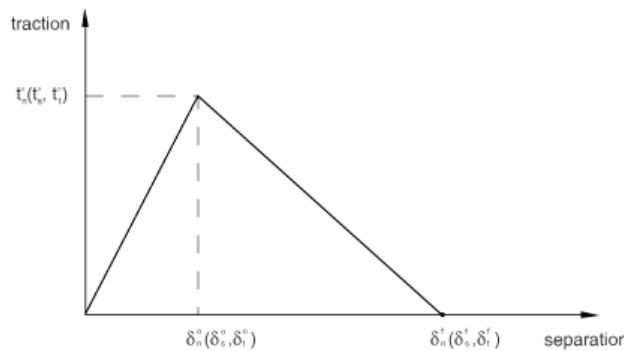


Figure 96: Typical traction-separation law

The linear elastic behaviour is described with a traction separation behaviour, by defining the factors E_{nn} , E_{ss} and E_{tt} such as:

$$t = \begin{Bmatrix} t_n \\ t_s \\ t_t \end{Bmatrix} = \begin{bmatrix} E_{nn} & E_{ns} & E_{nt} \\ E_{ns} & E_{ss} & E_{st} \\ E_{nt} & E_{st} & E_{tt} \end{bmatrix} \begin{Bmatrix} \varepsilon_n \\ \varepsilon_s \\ \varepsilon_t \end{Bmatrix} = E \varepsilon \quad (1)$$

where:

t : nominal traction stress vector

t_n , t_s and t_t peak stresses in normal, in-plane and out-of-plane direction

In the case of the interface, the behaviour is assumed to be identical in all three directions, so the coefficients are equal for each temperature.

For the damage initiation, a maximal nominal stress criterion is used. The damage occurs when the nominal stress in one of the three principal directions reaches the specified maximum value. This criterion can be written as:

$$\max \left\{ \frac{t_n}{t_n^0}, \frac{t_s}{t_s^0}, \frac{t_t}{t_t^0} \right\} = 1 \quad (2)$$

where:

t_n^0 , t_s^0 and t_t^0 being the nominal strengths in the three main directions

For the damage evolution, an exponential softening function based on fracture energy principle was used as after many trials it was found that this was the best way to approximate the actual behaviour.

7.1.5 Mesh

When choosing the mesh size for the model, some guidelines need to be considered. It applies, the finer the mesh, the better the results. However, convergence time increases significantly when the number of elements is increased, i.e. when their size is reduced. Further, the distribution of the elements can be adapted to concentrate the computational effort on the regions of interest. In this situation, the entire shear plane formed by the interface must be made up of thin cohesive elements. Since it is necessary to share the nodes for such elements, the number of elements in our model depends essentially on the number of cohesive elements. In fact, it was shown that the number of elements that constitute the thickness of each layer has no influence on the results, i.e. only one element is required for the thickness of each layer. Thus, the total number of elements in the model is 3 times the number of cohesive elements.

To determine a suitable mesh size, several trials with different mesh sizes were carried out. On one side, a very fine mesh was modelled in order to obtain very accurate results, resulting in a significant convergence time. Using this fine mesh, the force-displacement curve obtained is considered to be the most accurate curve possible. The same model was then calculated with increasingly coarser meshes. Each model is associated with:

- the maximum relative calculation error compared with the results of the most accurate model,
- the CPU time required to calculate the model.

The results are shown Figure 97. As the repeatability of the Leutner laboratory results is above 1%, the numerical modelling therefore assumes that the mesh is acceptable if the relative error is less than 1%. The parameters are therefore elements with an approximate size of 8 mm, giving a total of 1122 elements.

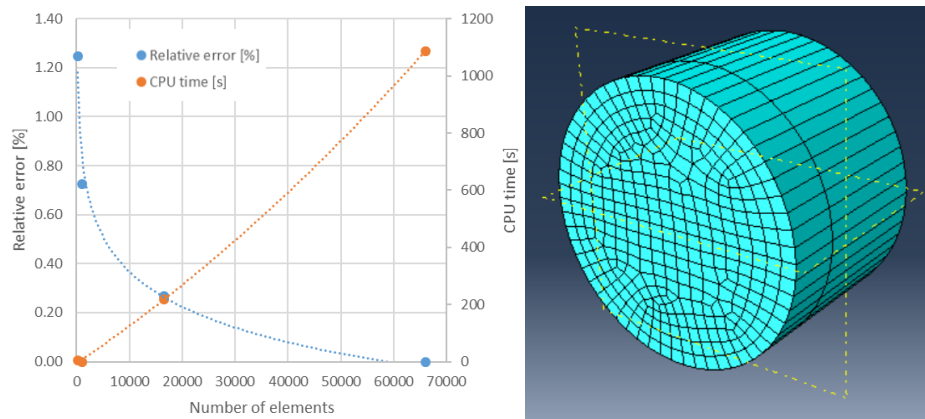


Figure 97: Relative error and CPU time vs. number of elements (left) and optimal mesh (right)

7.1.6 Load and boundary conditions

The fixed bottom layer is modelled with a fixed boundary condition in the three principal directions. The displacement is applied directly in a transverse and uniform way on the top layer. The value of this displacement corresponds to the standard value of 50mm/min for the Leutner test. Figure 53 shows these boundary conditions which fully describe the test.

It is possible to model the test mechanism so that it moves linearly during the test, but this places an additional load on the model and increases the computational power required for a purely visual effect. The main goal of the model is to remain simple so that the results can be more accurate in the areas of interest.

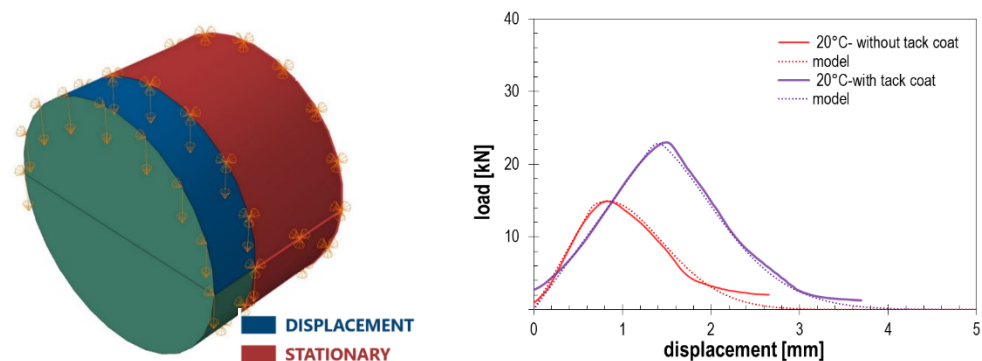


Figure 98: FEM geometry of the Leutner test (left) and comparison of the load displacement curves obtained with the test and the model

A temperature condition is included in the model, so the test can be run at any temperature. For the research project, a temperature range between -10 °C and 40 °C was chosen. Therefore, the model is only valid between -10 °C and 40 °C. Within this temperature range the values are linearly interpolated.

Regardless of temperature, a displacement of around 5 mm is sufficient to separate the asphalt layers during the laboratory test. For the numerical model, it is not necessary for the top layer to move by more than 5 mm, which corresponds to a test time of 6 seconds

7.1.7 Results

In order to make the model more accurate, no parameters for the interfacial behaviour are initially specified. The model is therefore subjected to repeated calculations to observe the influence of the parameters for the interface behaviour on the force-displacement curve. Depending on the temperature of the experiment to be modelled, the following parameters need to be determined:

- the elastic behaviour parameters E_{nn} , E_{ss} and E_{tt} , equal to each other
- the nominal strengths parameters t_n^0 , t_s^0 and t_t^0 , equal to each other
- the fracture energy G_c

Figure 99 to Figure 101 show the influence of the various parameters when they are changed independently. From these figures, it can be deduced that the elastic parameters have an influence on the slope before the peak, the nominal strength has an influence on the load value at the peak and the fracture energy has an influence on the area below the curve. Considering this, it is now possible to calibrate the model for samples without tack coat at each temperature. The model was also calibrated at the standard temperature of 20°C for samples with tack coat.

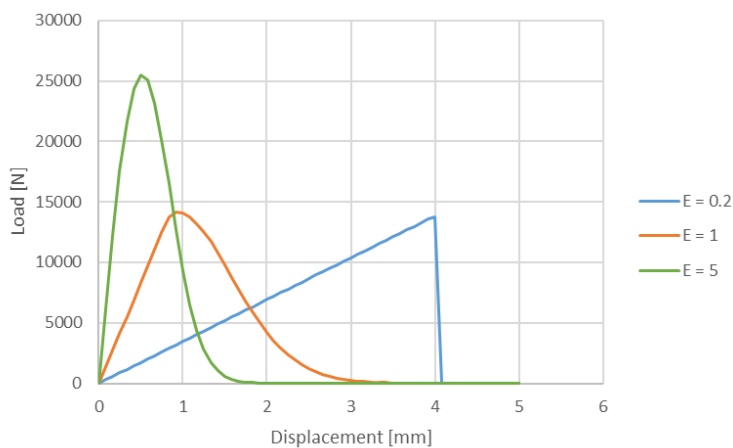


Figure 99: Influence of the elastic behaviour on the load - displacement curve for different elastic behaviour parameters (E)

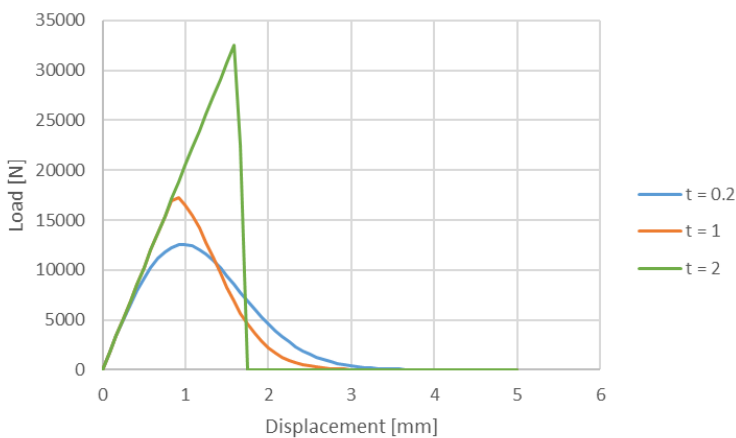


Figure 100: Influence of the fracture energy on the load - displacement curve for different nominal strengths parameters (t)

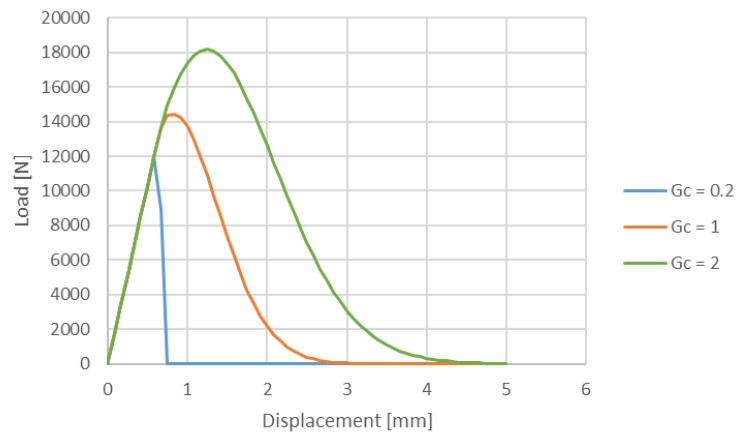


Figure 101: Influence of the fracture energy on the load - displacement curve for different fracture energy values (G_c)

The force-displacement curves from the laboratory tests used to calibrate the model and the model results using the optimal parameters, are shown in Figure 102 to Figure 105.

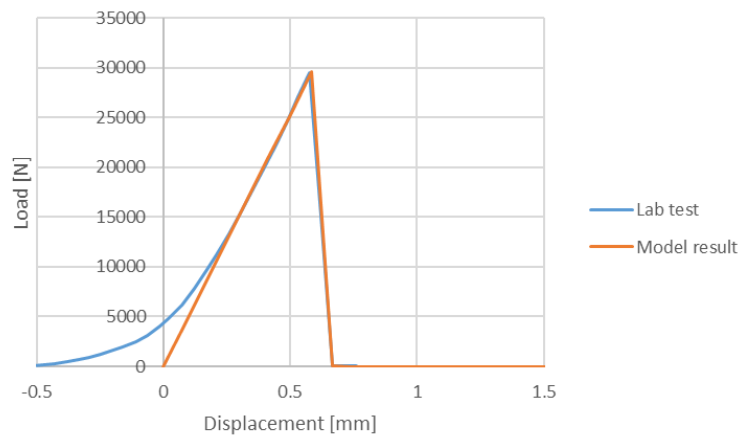


Figure 102: Model-lab test comparison for samples without tack coat at -10°C

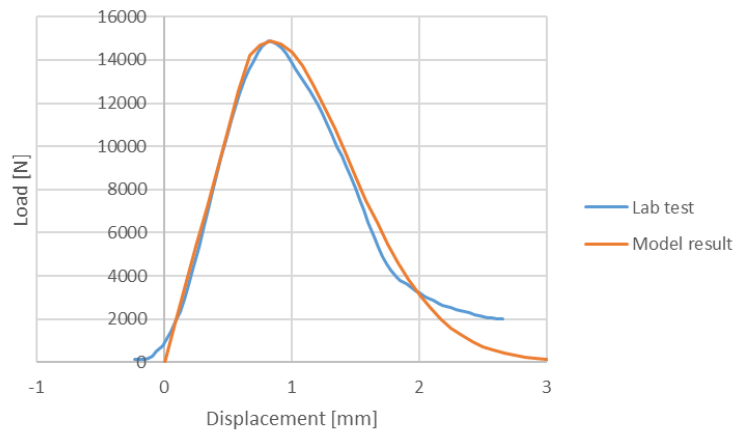


Figure 103: Model-lab test comparison for samples without tack coat at 20°C

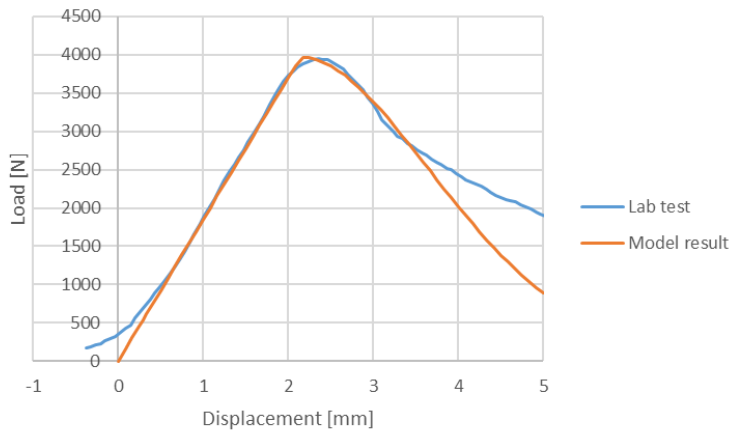


Figure 104: Model-lab test comparison for samples without tack coat at 40°C

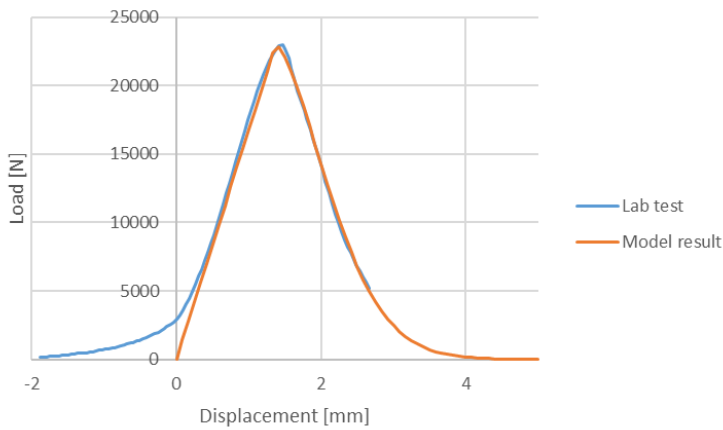


Figure 105: Model-lab test comparison for samples with tack coat at 20°C

The coefficients of the cohesive elements that best fit the laboratory results, as shown in previous figures, are presented in Table 12 to Table 14.

Cohesive elements modelling parameters: elastic coefficients

Temperature	Enn	Ess	Ett
-10 °C	3.02	3.02	3.02
20 °C	1.25	1.25	1.25
40 °C	0.107	0.107	0.107

Table 12: Elastic coefficients without tack coat

Cohesive elements modelling parameters: max. nom. stress

Temperature	Tn,o	Ts,o	Tt,o
-10 °C	1.92	1.92	1.92
20 °C	0.8	0.8	0.8
40 °C	0.225	0.225	0.225

Table 13: Maximal nominal stress value for the damage initiation without tack coat**Cohesive elements modelling parameters: fracture energy**

Temperature	Fracture energy
-10 °C	0.2
20 °C	1.1
40 °C	0.7

Table 14: Fracture energy values for the exponential softening

Table 15 and Table 16 provide a summary of the peak values with the difference between the model and the laboratory tests. Given the accuracy of the Leutner test, the results obtained with the model are clearly acceptable. The differences are greater the higher the test temperature, mainly because the scatter of laboratory tests increases significantly.

Model vs. lab comparison without tack coat

Temperature [°C]	Maximum load [kN]			Peak displacement [mm]		
	Lab	Model	Diff [%]	Lab	Model	Diff [%]
-10	29466.0	29564.1	0.33	0.575	0.583	1.44
20	14861.9	14886.1	0.16	0.818	0.833	1.91
40	3941.2	3962.8	0.55	2.355	2.167	8.00

Table 15: Model-lab test peak values comparison for samples without tack coat**Model vs. lab comparison with tack coat**

Temperature [°C]	Maximum load [kN]			Peak displacement [mm]		
	Lab	Model	Diff [%]	Lab	Model	Diff [%]
20	22860.9	22956.4	0.42	1.464	1.417	3.25

Table 16: Model-lab test peak values comparison for sample with tack coat

The results obtained here are to be implemented in the MMLS3 simulations to model the interlayer bonding behaviour, as explained next.

7.2 Modelling the MMLS3 tests

In this section, the effect of an asphalt reinforcement within the road structure is simulated with a numerical model. Specifically, the finite element method used in this work is based on the commercial ABAQUS software, but extended using the Python

API to design a plug-in with windows that allow the users to fill up fields to intuitively create the model. The plug-in consists of a user-friendly Graphic User Interface (GUI) accessible through the ABAQUS/CAE environment where the user can enter the data of the pavement and grids geometry, the temperature of the different layers, the interlayer interaction, loads, moving load speed, etc. The preparation of a model of this complexity would be otherwise (using CAE or writing an input file with the model data in from of a text file) difficult and time consuming, making any sensitivity analysis a tedious and time-consuming work reserved only to FEM and/or Abaqus specialists. Therefore, the aim is to simulate and evaluate the effect of the MMLS3 tire passing on the slabs as a preliminary step before extending it to a real pavement model in the future. Ideally, the model including the reinforcement grids should be as accurate as possible in order to be able to evaluate the real stresses and strains, which have an important role in the performance of the slabs. Further, the preparation of the models should be simple and easy to create though engineers that are not familiar with the "language" of finite element. The goal is to provide a rational framework for evaluating different scenarios when no experience is available, without the need of experimental setups for calibration and field validation. The plug-in also comprises a material and load library to make it easy to create models and perform a multivariable sensitivity analysis.

7.2.1 Overview

The Abaqus plugin displayed within CAE is in the form of a GUI form and can be recalled from the CAE menu, as shown in Figure 106. This GUI form was created in such a way that parameters are entered and filled in each field displayed by selecting the different tabs, as explained in next chapters. The data input is organized by filling in five different forms by selecting their tab labels, which are related to the different stages of the organization of the model. The names of the forms, which can be activated by selecting the corresponding tab, are:

- Geometry and Materials
- Element types + Mesh
- Grid reinforcement
- Loads
- Interactions
- Job + Visualization

After selecting the characteristics of the model with the form and pressing the continue button, the plug-in writes a text file with a specific formatting including all the data of the simulation. This file has an ".inp" extension that can be sent to the Explicit Abaqus solver. By default, the script automatically creates 3 steps for analysis. After the calculations are finished (for this type of complex simulation the calculations can last several hours for one single tire passing) the results can be visualized using Abaqus Viewer.

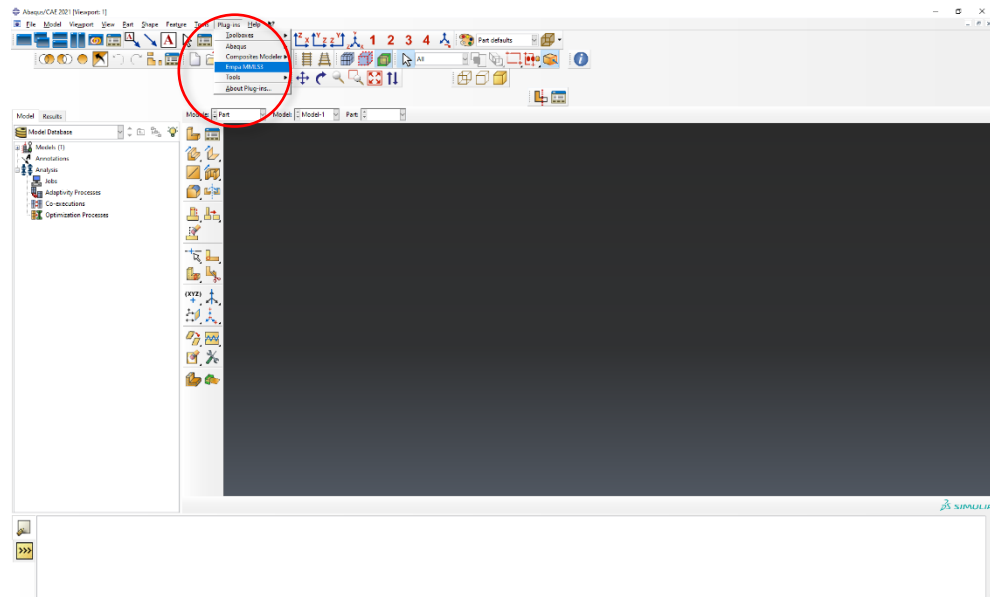


Figure 106: View of the Abaqus / CAE environment, where the plug-in for the MML3 can be called

7.2.2 Geometry and materials

Figure 107 shows the GUI form corresponding to the tab Geometry and Materials, where the geometry of the setup can be defined by choosing the width and length of the slabs. Further, the thicknesses and material properties of the layers can also be established. The geometry is defined through the position of the nodes that limit the slab, which are generated automatically by giving them a name and a position. The global origin of the coordinates system is set at the center of the lower right edge of the slab, from the loading direction perspective. From there, the Z-axis is perpendicular to the tire traveling direction and all layers are created in positive Y direction. The concrete slab is created automatically and is positioned in negative Y direction. The latter is established through a library, so the material can be selected with a dropdown menu from a database. In this case, the rubber was modeled based on the elastic properties according to the technical specifications and the concrete elastic values and Poisson ratio were obtained from the literature.

Layer	Thickness (mm)	Material
Layer 3 - ACB	A_carbon_fiber	ACB-REINF
Layer 2 - AC11	40	AC11-REINF
Layer 1 - Rubber	20	Rubber
Concrete	100 mm	

Figure 107: View of the form where the information about the geometry and materials of the model can be selected

The mixtures AC 8 S and AC B 11 S were modeled using the temperature dependent elastic modulus as described in § 7.1.4.1, but also with the temperature and time dependent viscoelastic properties as obtained from the tests described in § 5.1.

It is well known that asphalt materials used in flexible pavements are not purely elastic and the mechanical response of asphalt mixtures is dependent on time, type of loading and loading history. In general, the behavior of this type of material can be described by the theory of linear viscoelasticity. In the field of flexible pavement design, where loads are applied to the surface of a layered system over a subgrade, it is useful to analyze the viscoelastic response of the structure under these loading conditions. For each pavement material used in the finite element simulation, a constitutive law is required. The viscoelastic parameters were then derived from the test data as explained in the following.

The stiffness characteristics of asphalt mixtures depend mainly on the loading time and temperature:

$$S_{mix} = \frac{\sigma(t, T)}{\varepsilon} \quad (3)$$

where:

S_{mix} : mixture stiffness

σ : tension

ε : deformation

t : loading time

T : temperature

As explained in the previous section, the test provides the mechanical parameters in terms of the complex moduli E^* and phase angle ϕ . But asphalt mixtures can be characterised as a material with viscous behaviour at high temperatures and/or slow loading rates with a generalised Maxwell model. It can be assumed that volumetric stresses in the usual range of contact stresses of a wheel cause elastic and relatively small deformations of short duration, while deviatoric stresses are responsible for elastic as well as time-dependent deformations. Then, the stress tensor decomposed

into volumetric components and shear components can be modelled as linear elastic and linear viscoelastic. The linear viscoelastic behaviour can be represented by a generalised Maxwell model. Prony series can be used to fit the shear stress relaxation function of a viscous material using:

$$G(t) = \sum_{k=1}^K G_k e^{-t/\tau_k} \quad (4)$$

where:

$G(t)$: complex modulus calculated at time t

t, τ_k : relaxation time

G_k : Prony coefficients

K : number of elements of the generalized Maxwell model

In order to obtain the complex shear modulus, the following equation can be used:

$$G^*(\omega) = \frac{E^*(\omega)}{2(1+\nu)} \quad (5)$$

where:

$G^*(\omega)$: complex shear modulus for an angular frequency ω

$E^*(\omega)$: complex modulus for an angular frequency ω

ν : Poisson coefficient

In ABAQUS, the viscoelastic material is defined by a Prony series expansion of the dimensionless relaxation modulus:

$$g_R(t) = 1 - \sum_{k=1}^K g_k (1 - e^{-t/\tau_k}) \quad (6)$$

where:

$g_R(t)$: adimensional relaxation module

g_k y τ_k : material dependent parameters

In the case of time-dependent data, an analytical expression can be derived to convert the terms of the Prony series from the time domain to the frequency domain, using the Fourier transform:

$$G'(\omega) = G_0 \left(1 - \sum_{k=1}^K g_k\right) + G_0 \sum_{k=1}^K \frac{g_k \tau_k^2 \omega^2}{1 + \tau_k^2 \omega^2} \quad (7)$$

$$G''(\omega) = G_0 \sum_{k=1}^K \frac{g_k \tau_k \omega}{1 + \tau_k^2 \omega^2} \quad (8)$$

where:

ω : angular frequency
 $G'(\omega)$: storage modulus for angular frequency ω
 $G''(\omega)$: loss modulus for angular frequency ω
 G_0, g_k y τ_k : material dependent parameters

The model parameters G_0 , g_k and τ_k can be derived from the test data by minimizing the residual of the difference between the experimental data (storage and loss moduli for the test frequencies and temperatures) and those calculated with Eq. (4) and Eq. (5), using an optimization algorithm in Matlab and Excel. Table 17 shows the Prony series parameters that can be used to model the viscoelastic behaviour of AC 8 S and AC B 11S in Abaqus.

AC material Prony parameters for MMLS3 test modelling					
AC 8 S			AC B 11 S		
g_i	k_i	t_i	g_i	k_i	t_i
0.394	0.394	16.274	0.409	0.409	16.267
0.125	0.125	207.812	0.124	0.124	214.799
0.131	0.131	207.812	0.124	0.124	214.799

Table 17: Prony parameters used to model the viscous behaviour of AC 8 S and AC B 11 S

7.2.3 Reinforcement grids

The grids are linear elements that work only under tensile stress and are embedded within the pavement structure between the layers. They work, i.e. they start taking tension, when the pavement is deformed under load. Figure 108 shows the GUI form corresponding to the tab "Reinforcement grids" where the user can decide if the model has a reinforcement grid and defines the spacing between the grid reinforcement ribs, its cross-sectional area and materials in longitudinal and transversal direction. As for the asphalt concrete, rubber and concrete, the materials of the grids are also stored in a material database and can be selected from a table. The software then automatically defines the nodes of the grid considering that there is a rib in the center of the slab and counting their number based on the size of the slab and grid spacing.

Figure 108: Form for entering the information about the possible use of the grid and its geometry and materials is selected

7.2.4 Elements and mesh

The element used to define the slab layers is the solid cubic element with eight linear nodes C3D8R that uses linear interpolation and reduced integration technique for the calculations. On the other hand, for the grids the element type T3D2 truss was selected, and the mesh is defined by the nodes defined in chapter. As for the cubic elements, the form shown in Figure 109 is used to define the size of the elements. In the trade off resulting from long calculation times and more accurate results, it is recommended that the layers not important for the modelling have coarser mesh. Generally, for the lower layers (concrete and rubber) a coarser mesh is preferred, whereas it is recommended that the asphalt concrete layers have a finer mesh. In order to capture the tire loading in detail, it is recommended that the wheel path has an even more refined mesh. For all the simulations, the selected area size of the surface elements in the wheel path was 20 mm x 10 mm in length and transversal direction respectively. Only in the centre of the slab, the elements have a length of 5 mm, corresponding to the width of the notch.

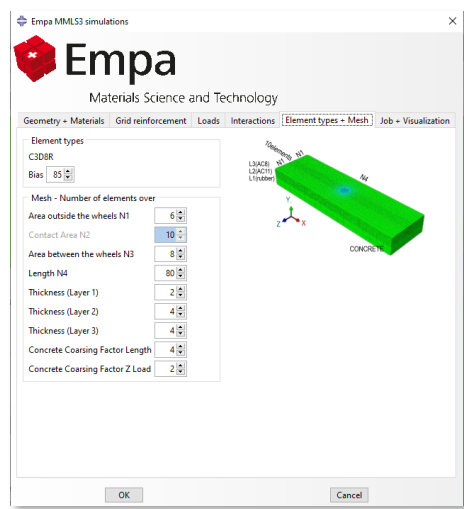


Figure 109: View of the form were the information about the meshing is entered

A view of the resulting geometry and meshes is presented in Figure 110.

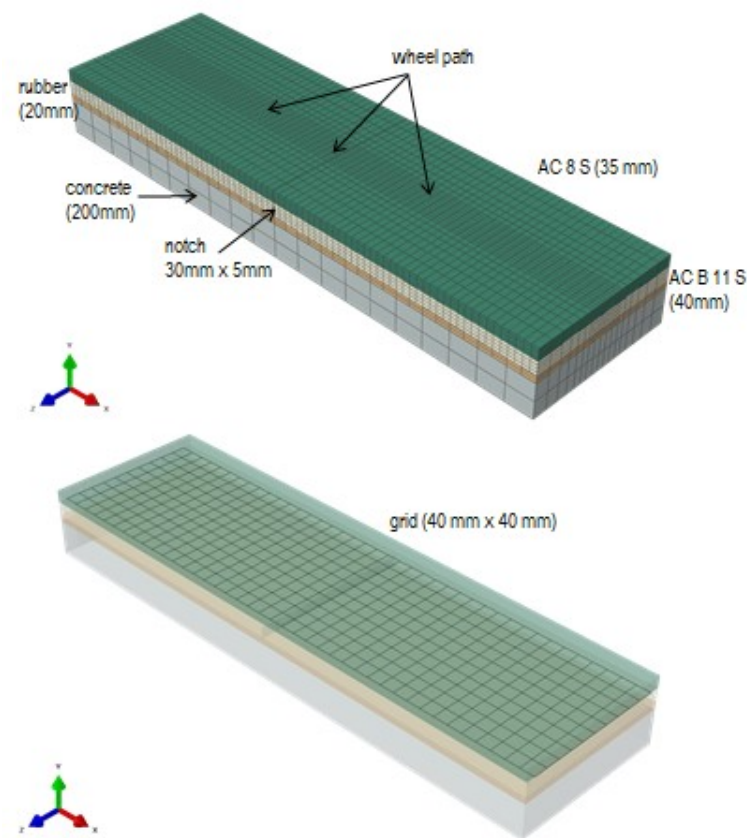


Figure 110: View of the geometry and mesh of the model

7.2.5 Load

The MMLS3 tires travel through the slab starting and finishing 30 cm after and before the borders, moving towards the positive X direction and covering a length of 1 m. To model this moving load, a time-dependent surface pressure function on each element in the wheel path is defined. As explained later in this chapter, the software automatically calculates a time-dependent amplitude function for each element based on the information provided in the GUI Loads form, as shown in Figure 111, where it is possible to select a load from a library, select its speed and the number of passings.

In order to prepare the load library and to define the dynamic loading of the MMLS3 tires as accurate as possible, a footprint was statically measured using different loads and contact pressures, by means of ink, a sheet of paper and a digital camera. Figure 112 a) shows a measurement for a 2.5 kN load and 6 bar tire pressure after using an image analysis program, showing in white the contact area of the tire. The shape was approximated using rectangular, circular and elliptical geometries and the average pressure was obtained using the load and the area (0.36 N/mm^2 in the case of the elliptical shape). Considering the size of the element's face, as mentioned in previous chapter, the tire footprint can be distributed in 10 elements in the transversal direction (later described as column number), as its width is approximate 10 cm. In the loading direction, the number and load on each element changes according to the time and the position of the tire. A total of 50 elements are involved in the calculations. The analysis about how the amplitude functions is briefly presented in Figure 112 b) and c) and will be presented elsewhere. Figure 112 d) presents an overview of the loading path, including some of the surface elements names required for the preparation of the loading functions.

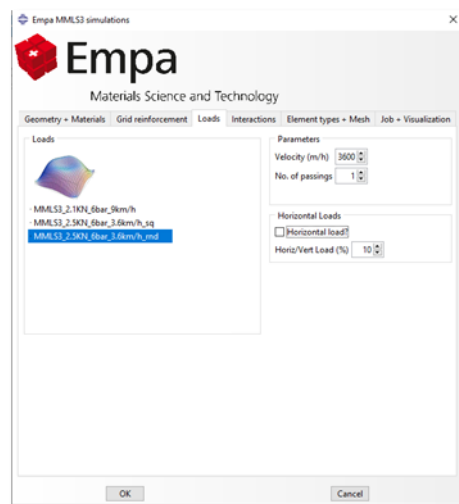


Figure 111: View of the load selection form

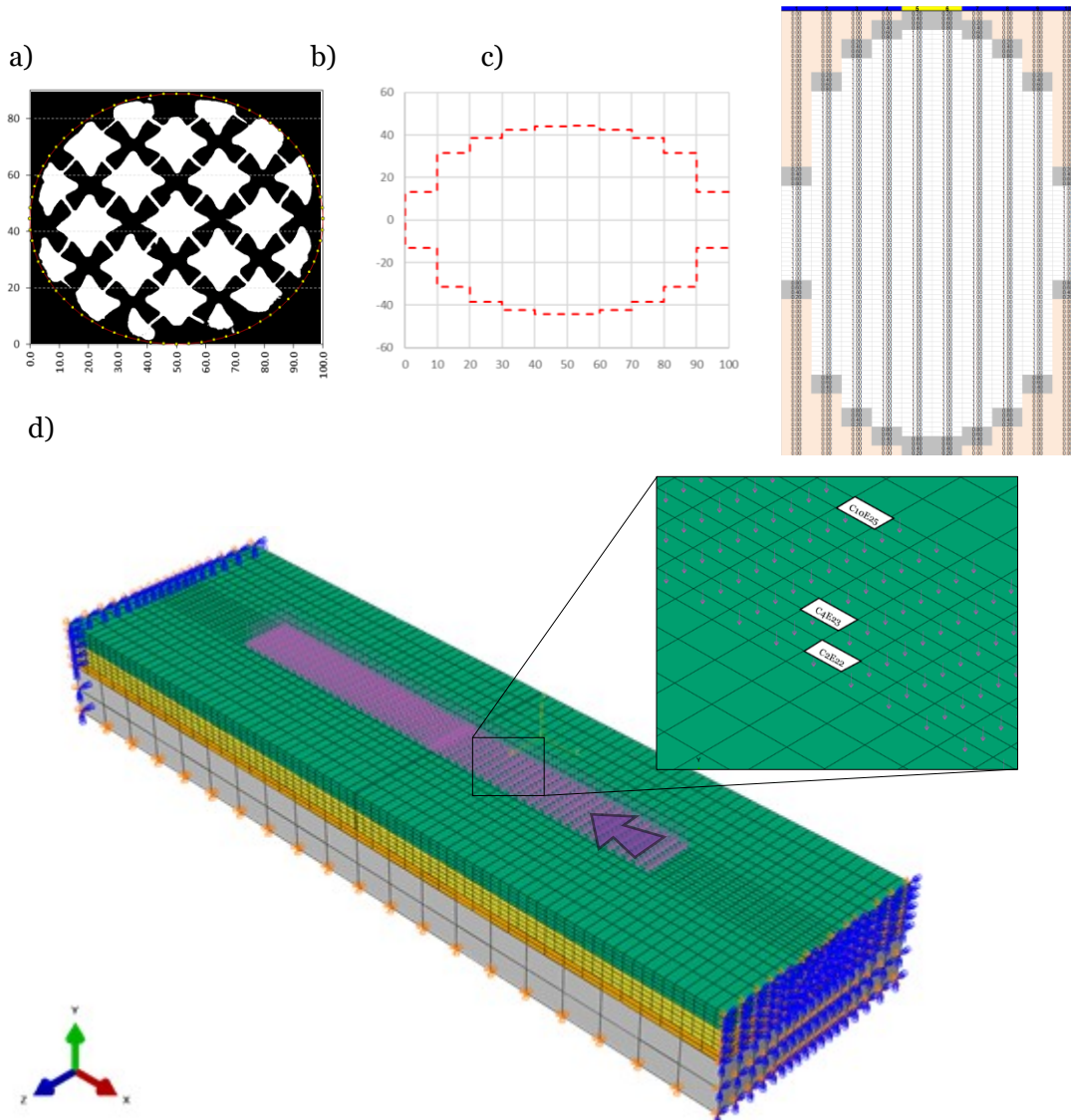


Figure 112: Different steps required for the calculation of the amplitude functions required to model a moving load

7.2.6 Boundary conditions

The slab has symmetrical boundary conditions on the short ends, and all elements in the bottom surface of the bottom layer are fixed in all directions, as shown in Figure 112d). Boundary conditions are created automatically by the software and cannot be changed.

7.2.7 Contact properties

The Interaction menu (see Figure 113) allows to choose the contact properties of the different parts of the testing slab. As a default, the model creates ties bondings, i.e. there is no relative displacement between the model instances and the nodes cannot penetrate or separate from neighbour elements.

If the tick is not on, it is possible to specify a friction coefficient in tangential direction and contact stiffness in normal direction using a contact pairs algorithm. Abaqus provides 4 options for pressure-overclosure in normal direction, namely, “Hard” contact, Exponential, Linear and Tabular. If value of 0 is supplied in the window, the

model would use hard contact properties as pressure overclosure and do not allow separation. Hard contact is the most common situation: when surfaces are in contact, any contact pressure can be transmitted between them. The surfaces separate if the contact pressure reduces to zero. It is the default and recommended setting. A value of more than 0 can also be specified, and then linear pressure overclosure would be used. Finally, if the checkbox for the cohesive elements is on, the program applies this interaction between the asphalt layers using the coefficients obtained in § 5.1.1.4. These coefficients are stored in a library and for the moment can only be changed by the programmer.

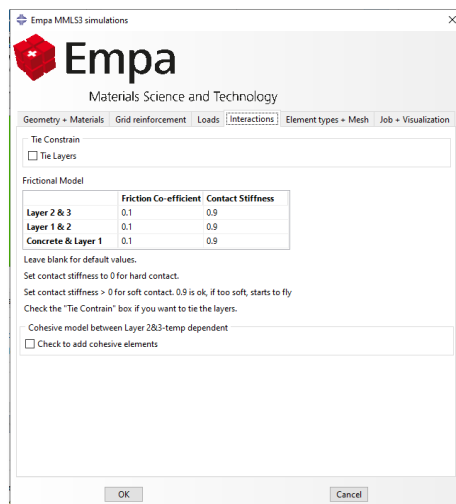


Figure 113: Contact properties form

7.2.8 Validation and results

The left side of Figure 114 shows a view of a model and the locations where the model responses were obtained. In order to do a calibration of the model, the displacements measured in-situ were compared to the displacements (U2) obtained in the model for node L1_9307, as this node is located in the same measuring position of Lvd1 3. The right side of Figure 114 present both values compared, after adjusting the elastic modulus of the rubber mat. Both curves, corresponding to a tire passing, are in a good match, and demonstrate the good accuracy of the model.

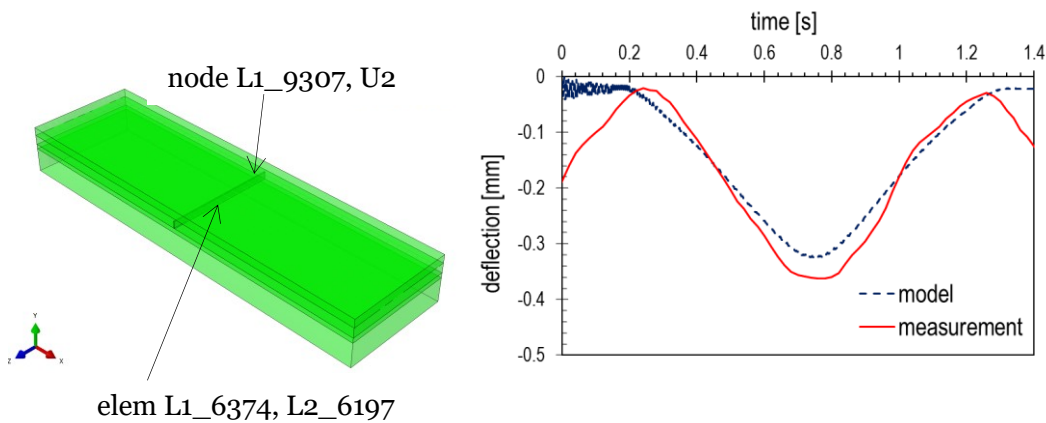


Figure 114: Node and elements used for the calibration of the model and to obtain the internal responses

Figure 115 shows the stresses in the longitudinal direction produced by the passing of a tire using a color map where the bluish and the red colors represent compression and tension respectively. The left side of the figure presents a result where no grid is present, whereas on the right side a grid of a glass fiber and 15 mm x 15 mm mesh opening is present. The rest of the parameters are the same. Through a simple comparison of both models, it is possible to observe that the stresses of the model with grid are smaller than the one without grid. These stresses are of particular importance, because in mechanistical-empirical design methods these tension stresses at the bottom of the asphalt concrete layer are related to the crack formation though fatigue tension. In Figure 116, the different strains are compared. It can be observed that there is a slight difference between them, the strains in the situation where a reinforcement grid is installed being slightly smaller. Figure 117 presents the strains induced in the grid though a passing tire, showing that the grid works to reduce the strain peaks in the asphalt concrete mainly though the longitudinal ribs.

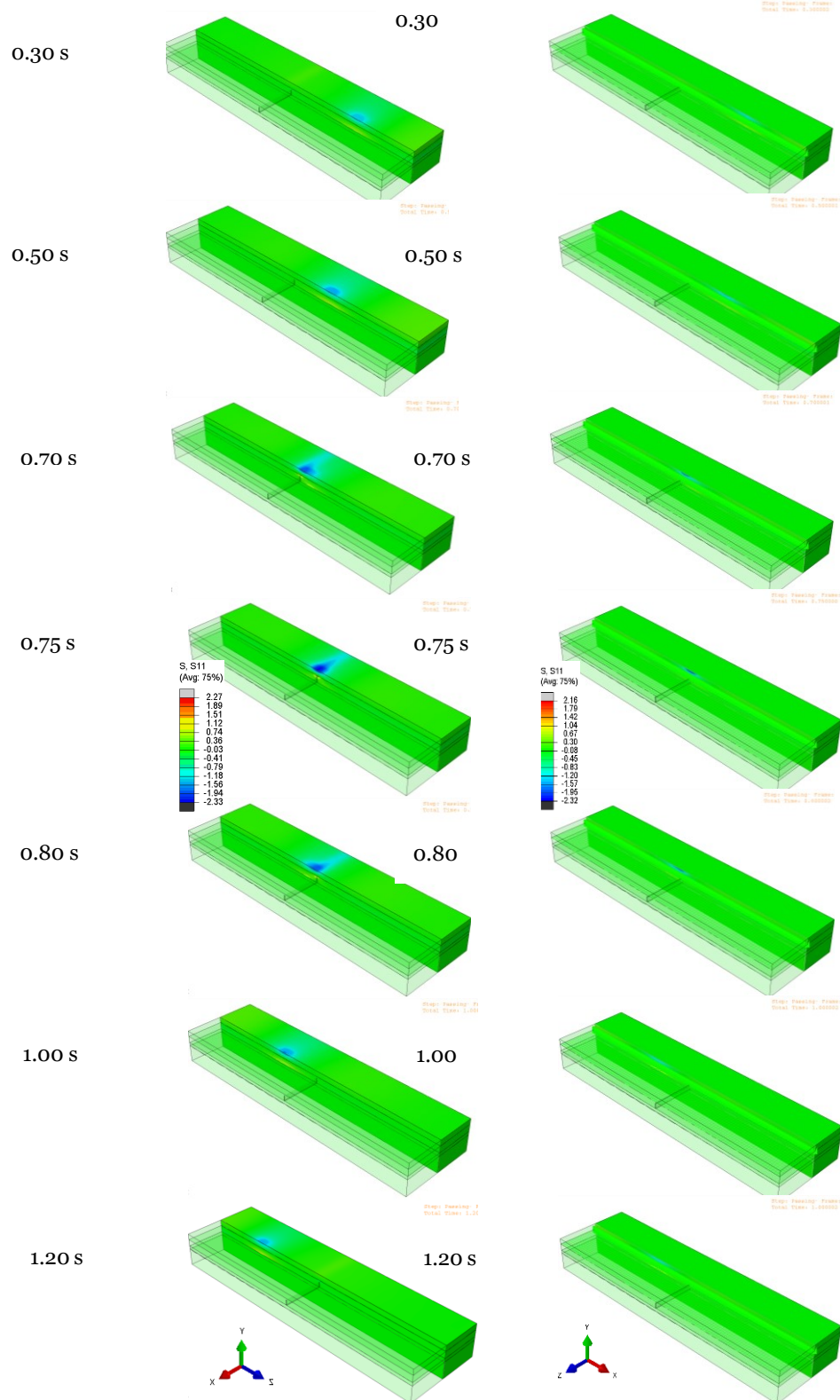


Figure 115: Longitudinal stresses produced by a tire passing at different timesteps for the model without (left) and with grid (right)

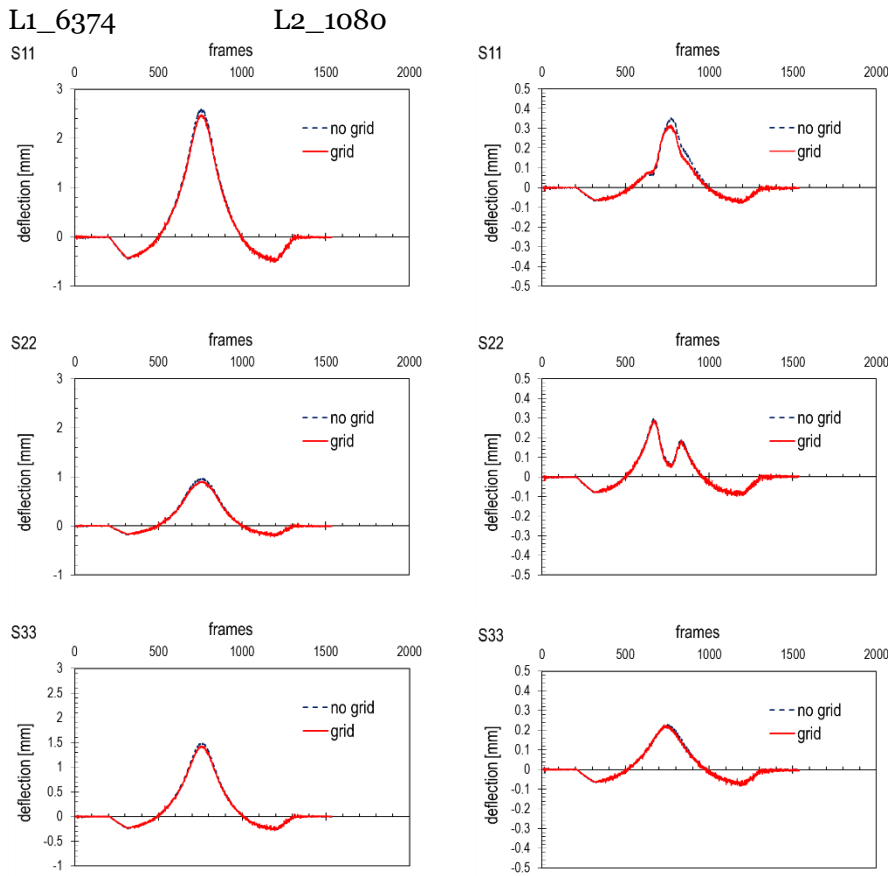


Figure 116: Strains in the 11, 22 and 33 directions for an element on top of the notch in the AC B 11 S layer L1 (left) and in the AC 8 S layer L2 (right) for the slab with and without grids

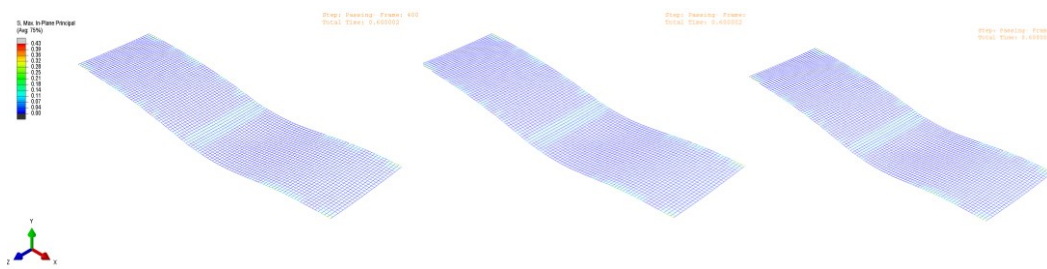


Figure 117: View of the strains in the reinforcement grid when a tire is passing

7.2.9 Outlook

The validation results presented earlier demonstrate the high accuracy and reliability of the developed model, establishing a solid foundation for further in-depth analyses incorporating the specific properties of reinforcement grids. To advance this work, it is essential to obtain detailed information about the geometric and mechanical properties of the grids, including the spacing between ribs, cross-sectional area, and the elastic modulus of the materials. Additionally, performing Leutner tests on samples with grids at a minimum of three different temperatures (as outlined in Section 6.1.1) is crucial. These tests will help determine the interlayer bonding properties of each grid and derive the cohesive element parameters required for integration into the pavement model.

Once these parameters are incorporated, the model can simulate various scenarios, such as changes in load magnitude, vehicle speed, ambient temperature, and the grid's position within the pavement structure. These simulations will enable a detailed evaluation of the effects of each grid type on the stress and strain distribution within the pavement system.

Furthermore, as previously emphasized, future work should extend the MMLS3 simulations to real pavement structures and traffic loads. Modeling a full-scale pavement under realistic loading conditions would provide critical stress and strain data, which are necessary inputs for mechanistic-empirical models used in the design and optimization of pavements reinforced with grids. This comprehensive approach would enhance our understanding of the long-term performance and potential benefits of reinforcement grids in mitigating reflection cracking and improving pavement durability.

8 Life cycle assessment

8.1 Introduction

In this chapter, a life cycle cost analysis (LCCA) is carried out to illustrate the benefits and associated costs of using asphalt reinforcement compared to conventional construction methods. To this end, the basics of a life cycle cost analysis are first outlined (chapter 8.2). This includes the determination of the temporal and spatial scope of the analysis as well as the definition of the benefit and cost components to be analysed (chapter 8.3). In this study, the benefits are quantified based on defined key performance indicators (KPIs) (chapter 8.4). The calculated costs refer to the pure measure-related costs without taking other external cost components into account. The methodology developed is tested using an application example and results are discussed (chapter 8.9).

8.2 Basics of LCCA

The term life cycle management is understood to mean the transformation process of infrastructure assets within the life cycle phases of planning, execution, commissioning, maintenance during the operation phase, dismantling and recycling of the materials. Following Pelzeter (2017) and Lehardt, Seiler, Gerdes, Bombeck & Lennerts (2020), the term life cycle management can be defined as a

"Combination of all technical and administrative measures as well as management measures during the life cycle of a unit with the aim of optimising use, resources and information across life cycle phases" (p. 15).

Based on the guiding principle of sustainable development, holistic life cycle management includes the following objectives over the entire period under consideration (Herrmann 2010):

- minimisation of risks
- minimisation of costs and optimisation of revenues
- minimisation of social and environmental impacts
- maximisation of overall asset effectiveness and value retention

For the infrastructure manager, it is relevant to determine the total costs that a specific infrastructure asset generates over all life cycle phases (Lüking, Schneider, Hafen & Albrecht 2014). This is accompanied by using so-called pavement management systems (PMS) to manage the network-wide maintenance of these assets over the entire life cycle with the help of the available financial resources. Figure 118 shows the individual life phases of an infrastructure object and the flow of information to the asset management System Analysis tools.

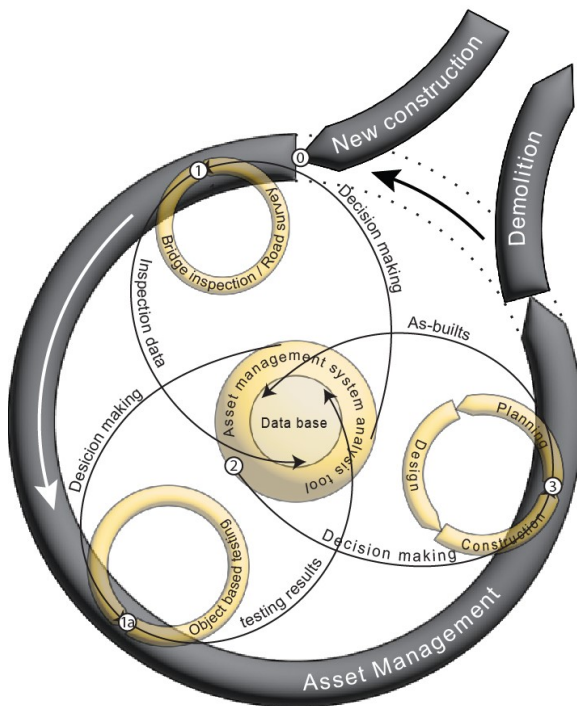


Figure 118: Life-Cycle of an infrastructure object

8.2.1 Planning and production

The planning of an infrastructure object takes place over several planning stages, in which the degree of detail in the elaboration of the design increases continuously. During the planning process, the individual elements of a road or a structure are worked out in increasing detail, from the initial alignment to the position of the road in the elevation plan to the construction of junctions, up to the actual detailed design. The individual planning stages build on each other, so that changes in previous planning stages always have an impact on the subsequent planning stage and planning elements must be adapted accordingly. In addition to the selection of materials for the construction of the road and the dimensioning of the road layers in accordance with the expected traffic load is carried out during the planning phase.

8.2.2 Construction

The construction phase includes all work associated with the construction of the infrastructure object. This includes setting up the construction site, carrying out preparatory measures, the actual process of building the materials and the final completion of the asset and opening it to traffic. The documentation of the construction process and the properties of the construction materials used and their composition for an infrastructure object, takes place within the framework of quality inspections. However, due to inhomogeneities and irregularities in the construction process, the result of the construction is no longer known in detail. The duration of use and the number of maintenance measures required vary with the quality of the materials used and the production process. Poor or inadequate workmanship leads to a shorter duration of the operational phase or requires more measures, whereas high-quality workmanship can lead to longer duration of the use phase or fewer measures. In the sense of an approach to life cycle management, the potential through inclusion of the

processes and determination of the planning and construction phase for the operational phase and the reflection of the findings of the operational phase into the next planning and operational phase is of great importance. In this way, costs can be optimised at an early stage over the entire life cycle of an infrastructure object.

8.2.3 Operational phase

After the release for traffic of an infrastructure object, the operational phase begins. Regular condition inspections are carried out as part of the long-term monitoring process and maintenance management, which allow conclusions to be drawn about the condition of the asset. The data collected must be evaluated regarding the achievement of previously defined goals, with the help of suitable key performance indicators. For the determination of performance indicators, measurements, non-destructive and non-destructive examinations can also be carried out.

8.2.4 Maintenance measures

The planning of measures and their implementation are assigned to the superordinate maintenance management. Maintenance management should be seen as only one part of life cycle management. It can be interval-based, condition-based or predictive. For condition-based or predictive maintenance knowledge of the current as well as the future condition of the asset is crucial for the selection of maintenance measures. Based on this information and other constraints, decisions are made on the type, extent, and timing of measures. Even though measures are applied during the operational phase, many problems originate already in the planning and construction phase. Short-term maintenance planning is based on in-depth investigations and structural analyses and includes a detailed specification of interventions to be undertaken in a timely manner. Medium to long-term maintenance planning is a process in which different intervention scenarios are developed over the longer time horizon. In general, one can distinguish between preventive, corrective and operative measures. The aim is to estimate the future financial and personal resource needs. Early planning makes it possible to choose the optimal time for measures and to reduce long-term costs.

8.2.5 Deconstruction and recycling

Deconstruction and recycling of the infrastructure assets materials occur, for example, when a road can no longer sustain future traffic volumes and must be replaced by a wider road section.

8.3 Modules of life cycle assessment

In the context of life cycle analyses of infrastructure assets, the System Boundaries must first be defined. DIN EN 15643 describes the framework conditions for the assessment of infrastructure assets, making it possible to compare the assessment results of different scenarios.

The following components of the life cycle can be considered:

- Planning phase (A0)
- Production Phase (A1-3)
- Construction phase (including measures) (A4-5)

- Operational phase (B1-8)
- End of life phase (C1-4)
- Benefits and loads beyond the System Boundaries (D)

In accordance with DIN EN 15643, in the first step it is necessary to define selected scenarios for the life cycle assessment. These scenarios should be described in a comprehensible manner, be realistic and representative, correspond to the current situation and technology and be aligned with the technical and functional requirements described by the functional equivalent. Figure 119 shows the different life cycle phases of an infrastructure asset and which aspects should be considered in the life cycle analysis according to DIN EN 15643.

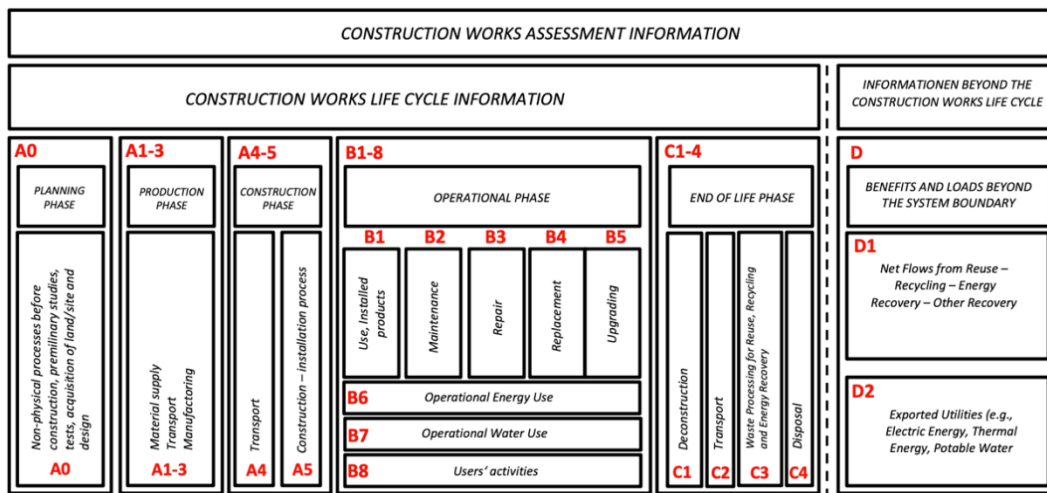


Figure 119: Construction works assessment information according to DIN EN 15643

8.4 Development of a key performance indicator system

Based on the literature research performed in the EU COST TU1406 Action (Hajdin et al. 2018) and the current state of management practice of road assets, it can be concluded that four key performance indicators (KPIs) that govern decision-making and that reflect the benefit of infrastructure to stakeholders are related to:

- reliability,
- safety for road users and road workers during the construction phase,
- availability,
- and sustainability.

It should be highlighted that values of KPIs of reliability and safety for the road users are here considered to change (i.e., worsen) over time due to slow, observable deterioration processes. On the other hand, the KPIs of availability and sustainability are affected by growth of traffic volume as well as by the planned interventions, i.e., predefined measures, which is described more detailed in the following sections. The main challenge remains to select the adequate PIs that will be used to measure the mentioned KPIs. Such PIs should account for the data that is already

collected/elaborated by infrastructure owners/operators (e.g., in traffic simulation, inspections, monitoring).

The economical KPI that is related to the costs of measures should be assessed separately, i.e., to reflect the effectiveness of maintenance interventions in improvement of the four KPIs. The associated costs can include planning, construction and operating costs as well as indirect (economic) costs. The linking of input data and calculation of performance indicators (PI) is done with the help of sub-models. Normalisation models are used for the conversion between PIs and KPIs. Figure 120 shows the developed KPI system.

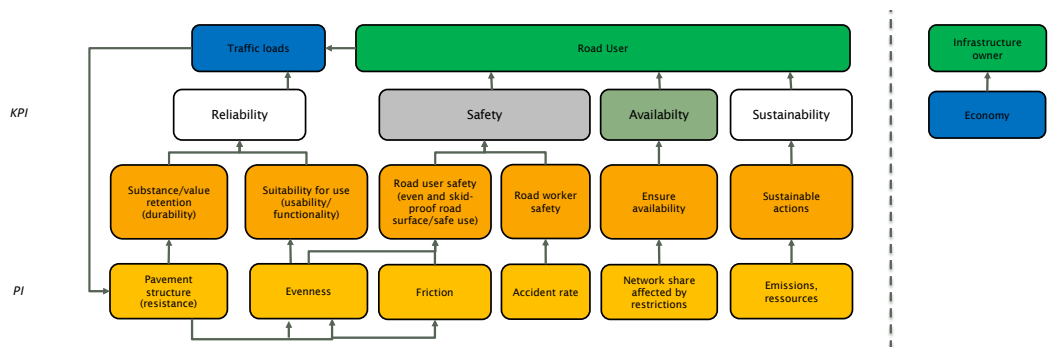


Figure 120: Key performance indicator (KPI) system with four KPIs – reliability, safety of road user and road workers, availability, and sustainability

8.4.1 Reliability

KPI reliability is the probability that an asset (e.g., a road section) will be functional during its operational phase. It is the inverse of the probability of structural failure (structural safety), operational failure (serviceability) or any other type of failure (e.g. fatigue), which is defined by infrastructure owner/operator provisions or given in standards and norms.

The following road condition Indices (Input data) are used to evaluate the KPI reliability:

- I0: surface distresses
- I2: longitudinal evenness
- I3: transversal evenness (rutting)
- I5: bearing capacity

KPI reliability includes two components – durability and usability/functionality. Both components are evaluated by using road condition surface indices. While durability is evaluated by road surface defects, bearing capacity, and cracking, the functionality of the road (suitability for use) is described by longitudinal and transversal evenness. Figure 121 shows the performance indicator system for determining the KPI reliability.

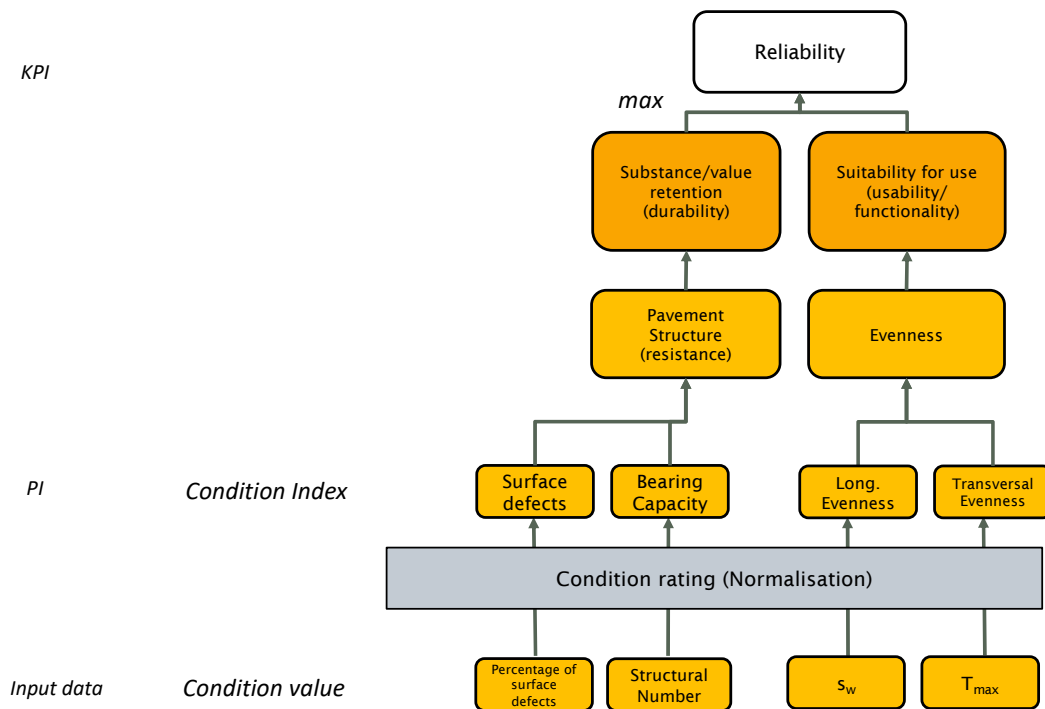


Figure 121: Normalization and linking of Performance indicators related to the KPI Reliability

The KPI reliability is calculated as the maximum of the condition indicators surface distresses (I_0), longitudinal evenness (I_2), transversal evenness (I_3) and bearing capacity (I_5):

$$KPI \text{ Reliability } (t) = \max(I_0(t), I_2(t), I_3(t), I_5(t)) \quad (9)$$

8.4.2 User safety

The KPI of user safety accounts for safety (of life and limb) for all road users (e.g., passenger, freight) during the operational phase. In this context, it is assumed that road users exercise reasonable caution, considering the external circumstances, and exercise the customary care. The most concrete PI to be used here is the accident rate or number of accidents. In this study, the following road condition indices (input data) are used to evaluate the KPI Safety during the operational phase:

- I_2 : longitudinal evenness
- I_3 : transversal evenness
- I_4 : skid resistance

Figure 122 shows the part of the key performance system for evaluating the KPI safety during the operational phase and the construction phase.

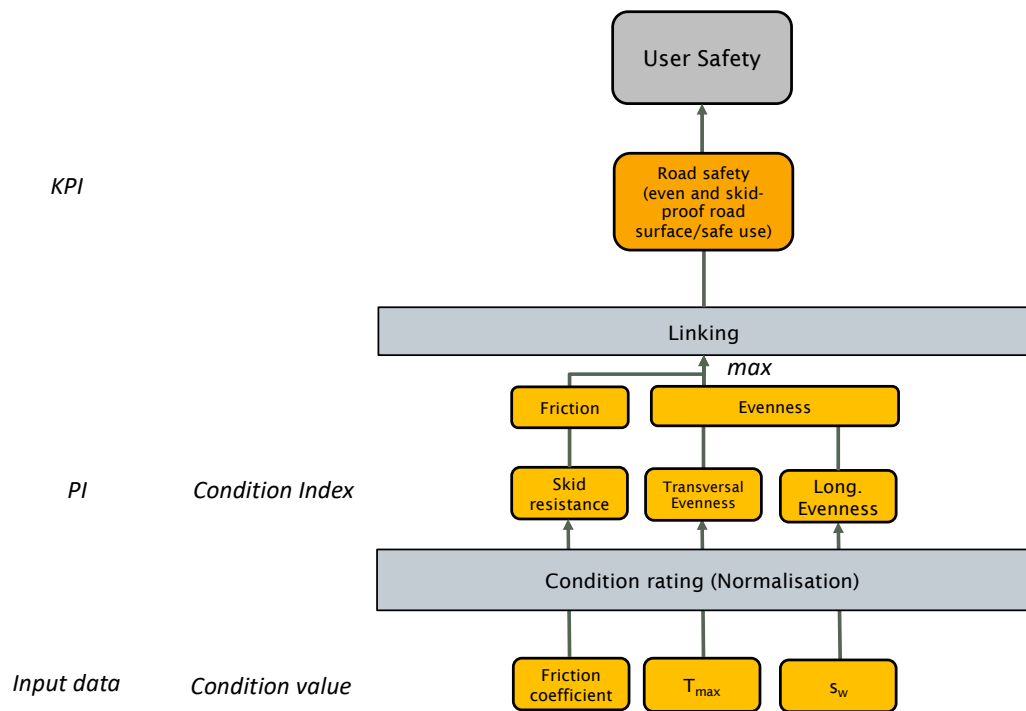


Figure 122: Normalization and linking of Performance indicators related to the KPI user safety

The KPI user safety is calculated as the maximum of the condition indicators longitudinal evenness (I2), transversal evenness (I3) and skid resistance (I4):

$$KPI \text{ user safety } (t) = \max(I_2(t), I_3(t), I_4(t)) \quad (10)$$

8.4.3 Availability

KPI availability refers to the proportion of time in each timeframe when an infrastructure asset fulfils its designated function. It is irrelevant here whether the asset's assigned functional failure is caused because of planned maintenance or as a result of a structural failure. However, it is essential to keep in mind that there can be different types of traffic restrictions, such as vehicle weight limitation, lane closures, and speed reduction leading to restricted use of the infrastructure object. Traffic restrictions can result in external costs for users in the form of additional expenditures due to loss of time (additional travel time), higher fuel consumption or higher accident rate (e.g., Schmellekamp 2016, Schiffmann et al. 2020).

The additional travel time is used to evaluate the KPI availability in this study and depends on measure type and extent since the duration of the measure as well as the temporary traffic control along the construction site and the corresponding route choice behaviour may vary. This data needs to be provided from traffic simulations. Figure 123 shows the working steps for determining the KPI availability.

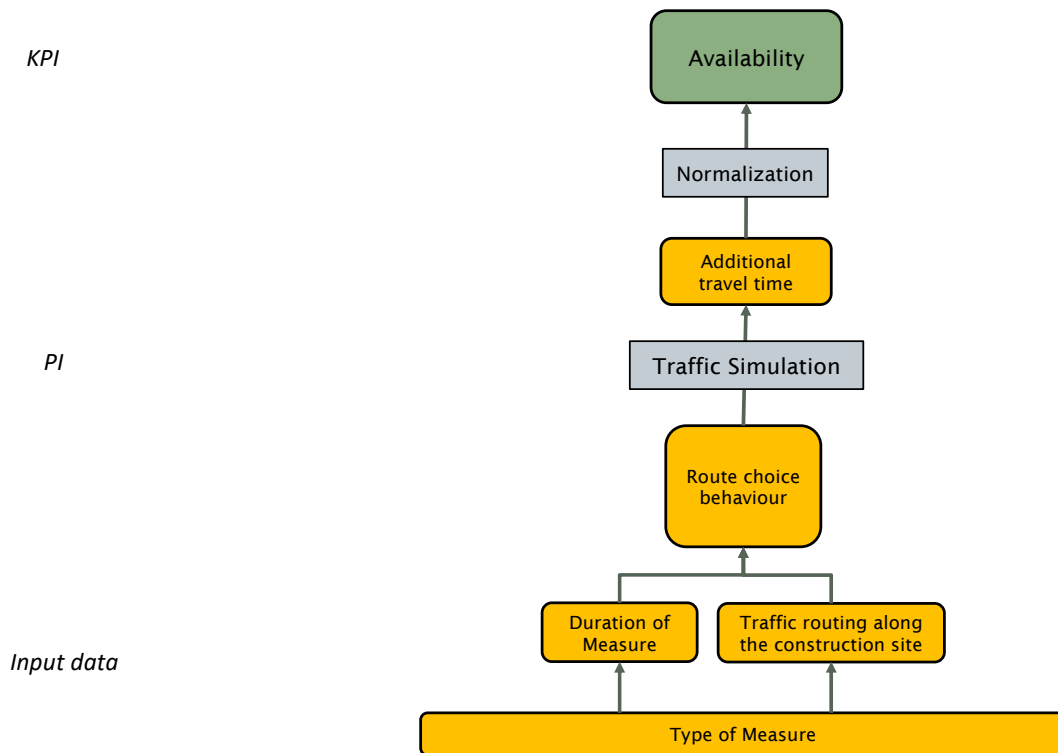


Figure 123: Normalization and linking of Performance indicators related to the KPI Availability

8.4.4 Sustainability

In general, the term sustainability encompasses environmental, sociocultural, and functional aspects. Environmental aspects include, for example, the possibility of recycling materials and minimizing the environmental impact during the service life of an asset. Sociocultural and functional criteria include, for example, the mobility needs, low noise pollution etc. However, there is no prevailing requirements in the literature on the content and significance of indicators for social aspects.

In this study indicators of the ecological sustainability dimension get considered to assess sustainability. The approach offers the user to carry out a sustainability assessment for an infrastructure object in relation to all life cycle phases.

To assess asset and road network-related emissions, the global warming potential (GWP, [kgCO₂-eq.]), the eutrophication potential (EP, [kgPO₄-eq.]), the acidification potential (AP, [kgSO₂-eq.]) and the summer smog formation potential (POCP, [kgC₂H₄-eq.]) can be used. All indicators are assessed in a “cradle to grave” scenario, i.e., considering the life cycle phases A1-5, B1-8 and C1-4 (see chapter 8.3). Construction site-specific data should preferably be used for their assessment. In the absence of such data, databases such as ÖKOBAUDAT or Ecoinvent can be used.

Figure 124 shows the general working steps for determining the KPI sustainability. The dashed lines within the diagram show potential indicators to be examined to evaluate the KPI. However, these were not taken into account in the modelling and the exemplary application.

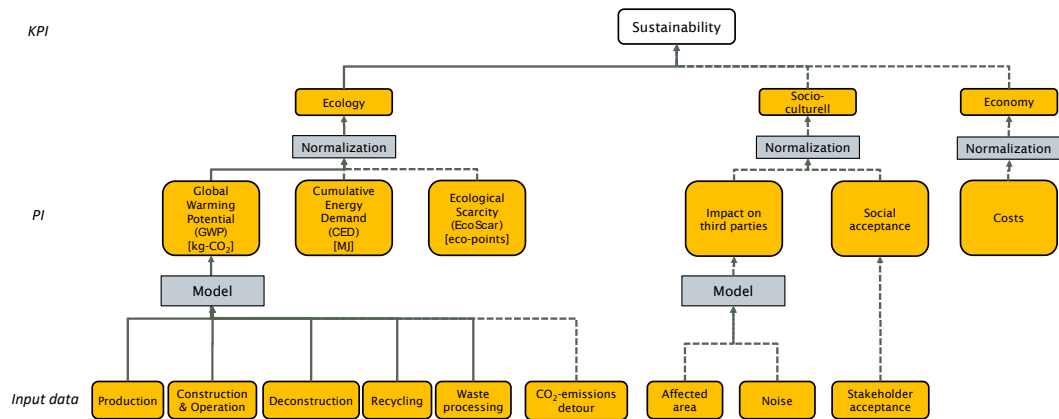


Figure 124: Normalization and linking of Performance indicators related to the KPI Sustainability

8.4.5 Economy

The performance target economic efficiency refers to the minimization of the long-term costs of measures, i.e., of measures over the life cycle of a road section, as most costs are incurred during the operational phase. The costs incurred can be allocated to different stakeholders. Costs can be divided into four cost components: infrastructure owner costs, accident costs, user costs, and environmental costs (see Figure 125). This study does not monetise any costs other than the infrastructure owner costs. This means that the effects on the safety, availability and sustainability KPIs (dashed lines in Figure 125) are not monetised.

The process-related infrastructure owner costs consist of the costs of the maintenance measures costs, the traffic control, and the work zone setup costs. The maintenance measure costs can be divided into material costs, machine costs and worker costs. All these cost components vary depending on the type of maintenance measure chosen.

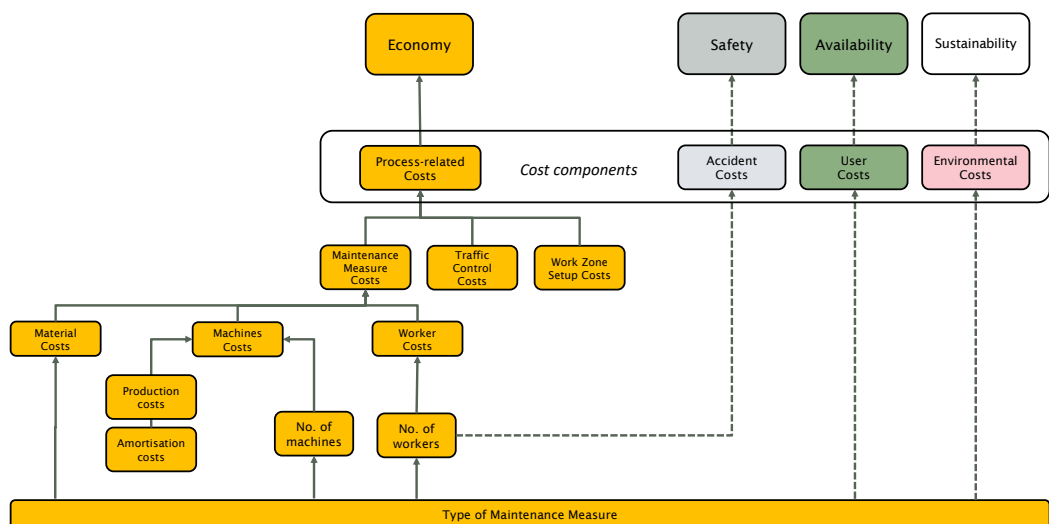


Figure 125: Submodel for evaluating process-related costs

8.5 Performance forecast models

Performance forecast models are used to forecast the change of performance indicators (PI) over time. The change of the KPI reliability and KPI user safety over time is not forecasted directly but via the individual road condition indicators (see chapters 8.4.1 and 8.4.2). The condition development of these indicators is forecasted using deterministic performance forecast models. The condition indicators can then be combined into a KPI for each time step. For the modelling of the performance curves the following general mathematical function was used:

$$PI(t) = a \cdot t^b + c \quad (11)$$

8.5.1 Performance curves of conventional asphalt pavements

The following coefficients were used in accordance with Scazziga (2008) for the performance curves of asphalt pavements without asphalt reinforcement:

Parameters of performance curves of conventional asphalt

Condition Index	Condition Indicator	Equation	Equation parameters		
			a	b	c
I0	surface distresses	$1,8 \cdot \text{age}^{**1,50}$	1,800	1,150	0,000
I1	surface distresses	$1,6 \cdot \text{age}^{**1,2}$	1,600	1,200	0,000
I2	longitudinal evenness	$1,753 + 0,07 \cdot \text{age}$		0,070	1,753
I3	transversal evenness	$3,061 + 0,539 \cdot \text{Log}(\text{age})$	0,539		3,061
I4	friction	$0,626 - 0,001 \cdot \text{age}$		-0,001	0,626
I5	bearing capacity	$4 \cdot \text{age}$		4,000	

Table 18: Parameters of performance curves of conventional asphalt

8.5.2 Performance curves of reinforced asphalt pavements

The use of asphalt reinforcement can increase the service life by approx. 20%, provided that the installation is technically correct. On the other hand, if the installation is carried out incorrectly, this can also lead to a reduction in service life of 20% compared to the reference variant (no use of asphalt reinforcement). The results of the MMLS 3 tests were used as the data basis for this. The performance curves of the pavements with asphalt reinforcement are shown in Table 19.

Parameters of performance curves for reinforced asphalt pavements

Condition Index	Condition Indicator	Equation	Equation parameters		
			a	b	c
I0	surface distresses	$1,7 \cdot \text{age}^{**1,300}$	1,700	1,300	0,000
I1	surface distresses	$1,45 \cdot \text{age}^{**1,35}$	1,450	1,350	0,000
I2	longitudinal evenness	$1,6 + 0,05 \cdot \text{age}$		0,050	1,600
I3	transversal evenness	$3,2 + 0,500 \cdot \text{Log}(\text{age})$	0,500		3,200
I4	friction	$0,626 - 0,001 \cdot \text{age}$		-0,001	0,626
I5	bearing capacity	$4,5 \cdot \text{age}$		4,500	

Table 19: Parameters of performance curves for reinforced asphalt pavements

8.6 Normalization and KPI rating

PIs can have different native units and be of different importance in the assessment of a KPI. Normalization involves the process of scaling the PI's native units on a scale of 1 to 5 (1 being the best score, 5 being the worst score). It is therefore used to make a uniform comparison based on assessment parameters. In the case of two or more KPI parameters, there are two possibilities for normalisation. In the first possibility, each parameter is scaled from 0 to 5 and the maximum is used as the PI value. In the second case, each parameter is scaled and the weighted sum results in the KPI value. Figure 126 shows an example of a normalisation function for longitudinal evenness for converting a condition state index (s_w -value) into a condition note (I2) in accordance with VSS 40 925b.

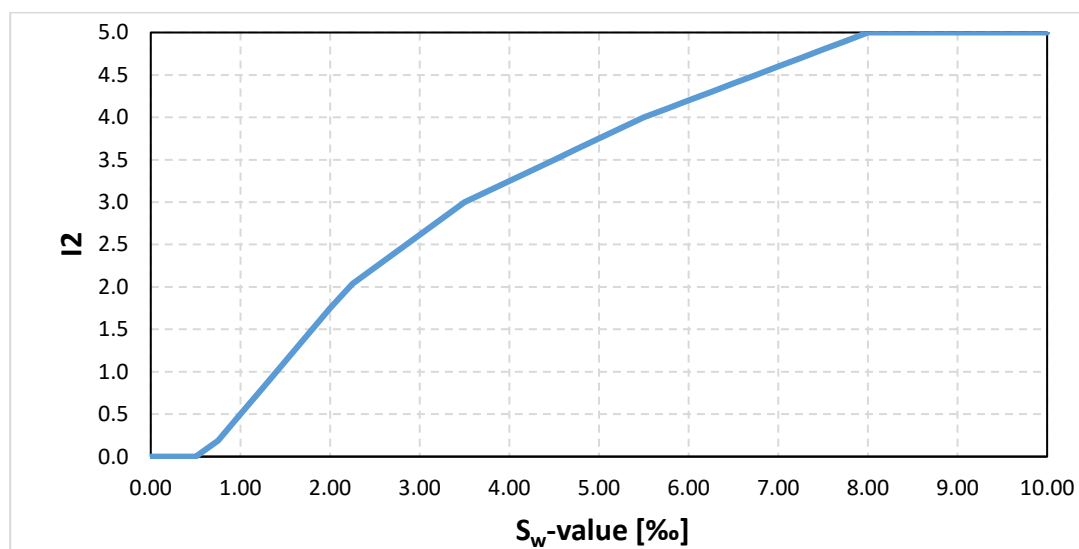


Figure 126: Example of a normalization function for the PI Longitudinal evenness (I2)

8.7 Measures

Measures are being carried out on the road to restore the originally planned level of serviceability. In principle, there are various types of maintenance measures, e.g. replacing the surface layer (I1/I2), replacing more than the surface layer (E1/E2), available, which differ in terms of extent, effect on damage repair and costs. The sequence of measures should generally be planned over a longer planning period and forms the basis for the comparison of different measure scenarios to estimate the long-term costs.

An analysis period of 50 years is used to compare a conventional maintenance scenario with the maintenance scenario including reinforced asphalt. Figure 127 shows a typical maintenance scenario including the sequence of measure types over the life cycle of a road section without reinforced asphalt.

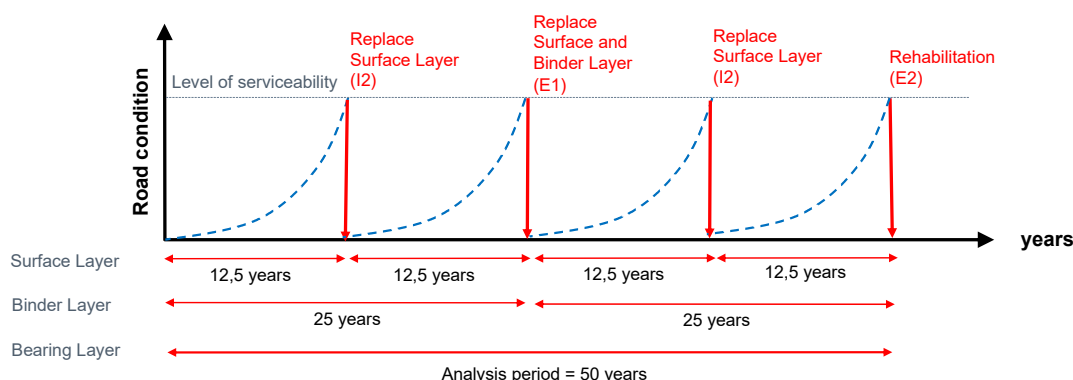


Figure 127: Typical maintenance scenario for the LCCA

The planning of measure types for individual road sections takes place at the object level. This should be done based on the values of the KPIs reliability and safety of road users and road workers. The aim of the chosen type of measure is to improve these

KPIs. The costs of measures over time must be also considered, as well as the impact on the KPI availability and sustainability. The realisation of a measure may lead to reduced availability of a single road section as well as of a whole part of the road network (additional travel times due to congestion and diversions). Furthermore, measures have negative effects on sustainability. One effect is the emissions caused during the production process of materials and the paving process. Another effect occurs during the construction phase and is reflected in increased pollutant emissions due to longer travel distances. These impacts are considered, allowing scenario analysis of maintenance strategies and helping in decision-making based on the assessment of KPIs. Purely administrative measures such as speed reduction, load limitation and lane closures to stop the further development of road surface damages or to reduce the potential number of accidents are not considered within this study.

In dependence on the type of measure the corresponding properties values may vary. Table 20 shows the properties for three relevant maintenance measures for asphalt pavements: replacement of surface layer (I2), replacement of surface and binder layer (E1), replacement of surface, binder and bearing layer (E2). These include the production costs per m³, the construction costs per m³, the deconstruction costs of existing pavements per m³, the recycling costs of the material, the transportation costs, and the productivity rate which reflects the duration of each measure in dependence on the actual size of the construction site.

Costs and duration of different types of measures

Properties	Type of measure		
	Replacement of surface layer (I2)	Replacement of surface and binder layer (E1)	Replacement of surface, binder, and bearing layer (E2)
Production costs (CHF/m ³)	217	359	479
Construction costs (CHF /m ³)	308	527	704
Deconstruction costs (CHF /m ²)	0.80	2.36	5.35
Recycling costs (CHF /m ³)	50	50	50
Transportation costs (CHF /km)	2.23		
Productivity rate* (km/24h)	7	4	1

*Duration depends on individual length of road section

Table 20: Costs and duration of different types of measures

Measures have the following effects during resp. after application on object level and network level.

Object level:

- Repairing of damages and thus improving the condition of the road section
- CO₂ emissions in the process of producing construction materials, the construction process including maintenance measures (paving, transport of materials) as well as recycling.

CO₂, ADPE and ADPF equivalents for different materials and life cycle phases

Life cycle phase	Construction material	CO ₂ eq.	ADPE eq.	ADPF eq.	Unit
Production	A1-A4 AC T	188.548	0.03024	7,682,350	kg/m ³
	A1-A4 AC B	206.276	0.03641	10,075,300	kg/m ³
	A1-A4 AC D	202.937	0.03787	10,613,250	kg/m ³
	A1-A4 Emulsion	0.166	0.00201	37,154	kg/LKW km
Construction & maintenance	A5 AC T	1.700	0.00015	23.40	kg/m ³
	A5 AC B	1.918	0.00017	26.40	kg/m ³
	A5 AC D	2.223	0.00019	30.60	kg/m ³
	A5 Cleaning	1.749	0.00015	23,478	kg/m ²
	A4 Transport Asphalt	1.345	0.00012	18,060	kg/LKW km
	A4 Transport Emulsion	2.287	0.0002	30,702	kg/LKW km
	C4 AC T	-69.722	-0.0197	-5,114.50	kg/m ³
	C4 AC B	-69.722	-0.0197	-5,114.50	kg/m ³
	C4 AC D	-69.722	-0.0197	-5,114.50	kg/m ³
	C3 Deposit	36.146	0.00366	512.80	kg/m ³
Recycling	C4 Preparation	7.057	0.00001	134.00	kg/m ³
	C1 Emulsion	12.2	–	–	kg/m ²
	C1 Deconstruction AC T 14 cm	1.08689	–	–	kg/m ³
	C1 Deconstruction AC B 8 cm	0.35488	–	–	kg/m ³
	C1 Deconstruction AC D 4 cm	0.08872	–	–	kg/m ³
	C1 Cleaning	1.749	0.00015	23,478	kg/m ²
	C2 Transport mixing plant	1.345	0.00012	18,060	kg/truck km
	C2 Transport landfill	1.345	0.00012	18,060	kg/truck km

Table 21: CO₂, ADPE and ADPF equivalents for different materials and life cycle phases (Mollenhauer et al. 2023)

Network level:

- Traffic disruption
- additional travel times along the construction site and within the road network
- additional CO₂ emissions (due to additional travel routes)

Table 21 shows the global warming potential (CO₂ equivalent), abiotic depletion potential for non-fossil resources equivalent (ADPE) and abiotic depletion potential of

fossil fuels equivalent (ADPF) for different types of construction material and life cycle phases according to DIN EN 15643.

8.8 Benefit-Cost-Analysis

By comparing maintenance scenarios (e.g. a sequence of measures) at arbitrary points in time, the effects on KPIs can be continuously analysed. Ideally, all indicators are completely fulfilled so that the area under each KPI curve is filled to the maximum. However, since this condition is practically impossible to achieve, evaluations can be made based on the differences between the scenarios considered and the ideal case. In general, the strategy that results in the largest sum of all areas under KPI curves over the analysis interval is considered as the one with the highest benefit, but the associated costs must also be considered in decision-making.

The Benefit of each scenario is calculated by the sum of areas under the KPI curves over time that is spanned by the four KPIs:

$$A_{V1} = \sum \left(\int_{t_1}^{t_2} R_1(t) dt, \int_{t_1}^{t_2} U_1(t) dt, \int_{t_1}^{t_2} A_1(t) dt, \int_{t_1}^{t_2} S_1(t) dt \right) \quad (12)$$

8.9 Use cases and results

8.9.1 Scope and framework

The subject of this application case is an urban road section with a total length of 300 meters and a width of 7.5 meters. The thickness of the asphalt layers and the resulting material masses with an assumed asphalt density of 2.400 g/cm³ are summarised in **Table 22**.

Basic data of the analyzed test sections

Property	Material				
	AC DS	AC BS	AC TS	Grid	Emulsion
Length [m]	300	300	300	300	300
Width [m]	7,5	7,5	7,5	7,5	7,5
Thickness [cm]	4,0	5,0	13,0	–	–
Volume [m ³]	90	113	293	–	–
Area [m ²]	–	–	–	2.250	2.250
Mass [t]	234	292,5	760,5	–	–

Table 22: Basic data of the analysed test section

The assumptions shown in Table 23 were used to calculate the transport routes and the resulting costs as well as the CO₂ equivalents. The use of articulated lorries with a payload of 20 tons per lorry was assumed for the delivery of the materials.

Assumed distances between production and construction sites and the dump

Route	Distance [km]
Production – Construction site	20
Delivery of grids	100
Delivery of emulsion	50
Construction site – dump	50

Table 23: Assumed distances between production and construction sites and the dump

An analysis period of 50 years is selected for the comparison of different maintenance scenarios. The existing road section, which has already been in place for some time, serves as the starting point. The surface damage shows a pronounced crack pattern, so that the current surface condition requires at least the replacement of the surface course. The underlying binder course and base course also show cracks. The age of the base course in this application is 37 years at the starting point of the analysis.

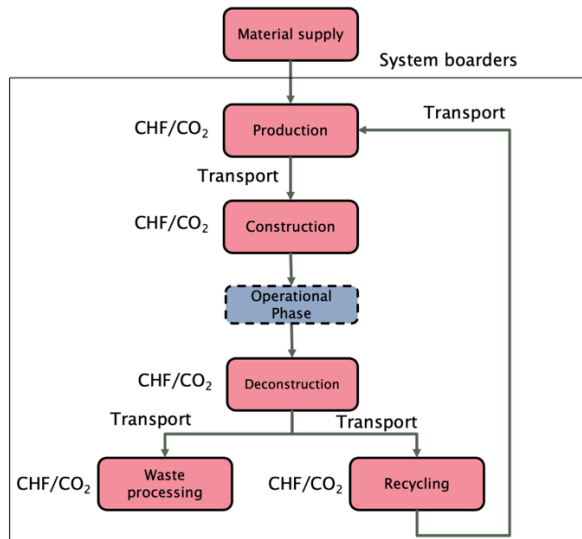


Figure 128: Process steps included in the Life-cycle analysis

As part of the maintenance planning, a decision has now to be made as to which maintenance strategy should be selected and how the selection of the type of maintenance measure will affect the further development of the road condition and the associated costs. A total of three model variants were examined, which differ mainly in terms of the type of measure at the beginning of the analysis period:

- 1) Reference variant: Replacement of all asphalt layers without the use of asphalt reinforcement

- 2) Variant 1: Replacement of the asphalt surface layer with the use of asphalt reinforcement with optimum paving quality
- 3) Variant 2: Replacement of the asphalt surface layer with the use of asphalt reinforcement with poor paving quality

The reference variant consists of a static interval maintenance approach. This includes every 14 years the replacement of the surface layer, and every 28 years both the surface and binder layers. The bearing layer is replaced every 56 years. This leads to a rehabilitation at the beginning of the analysis period. After 14, 28, and 42 years the surface layer gets replaced. After 28 years the binder layer gets replaced. Figure 129 shows the performance curves of the road surface condition and the sequence of measures of the reference variant over the defined analysis period of 50 years.

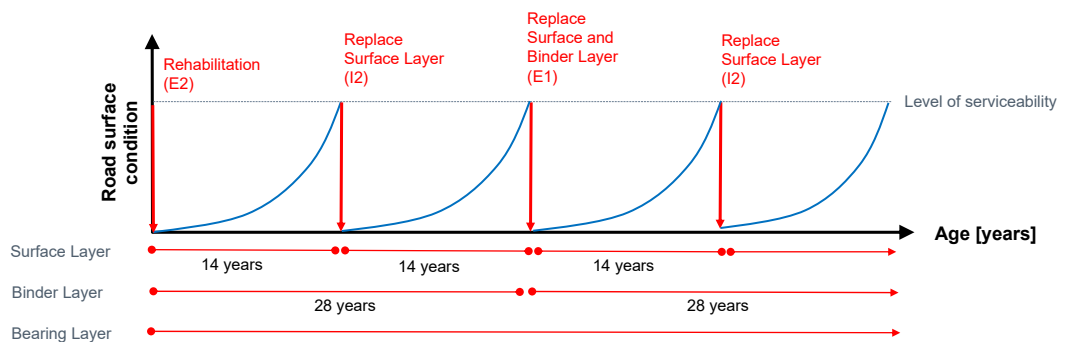


Figure 129: Sequence of Measures for the reference variant (conventional pavement)

Variant 1 consists of the use of reinforced asphalt with optimum paving quality. Instead of replacing all layers, only the surface layer gets replaced by a reinforced asphalt (I2). This leads to a postponed rehabilitation that is applied 10 years after the I2 measure. After the applied rehabilitation a static interval-based maintenance approach including another I2 measure, and the replacement of the surface and binder layer (E1) is applied. Figure 130 shows the performance curves of the road surface condition and the sequence of measures of variant 1 over the defined analysis period of 50 years.

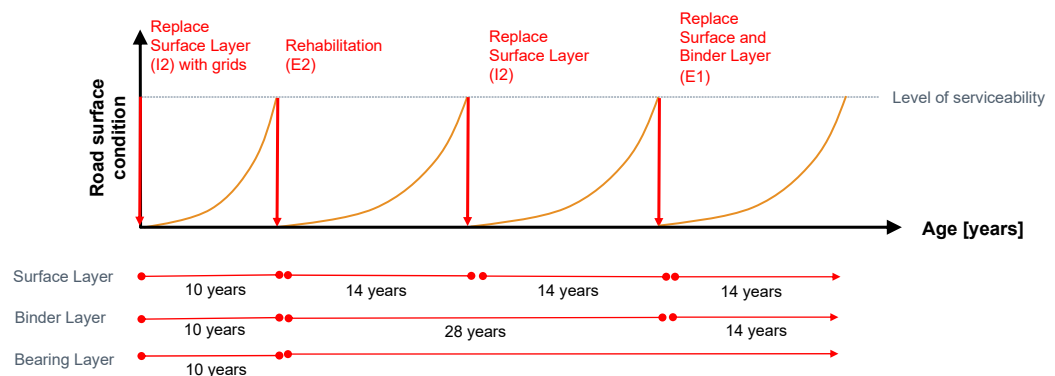


Figure 130: Sequence of Measures for variant 1 (asphalt reinforcement with optimum paving quality)

Variant 2 with asphalt reinforcement is based on the scenario that an optimum sheer bond between the asphalt surface course and binder course was not achieved during paving. As a result, the loads cannot be optimally transferred to the subsoil and

premature fatigue of the asphalt surface course occurs. As a result, cracks can be seen in the asphalt surface layer again after only a short period of use. This results in the premature replacement of all asphalt layers. The sequence of measure types for variant 2 is shown in Figure 131.

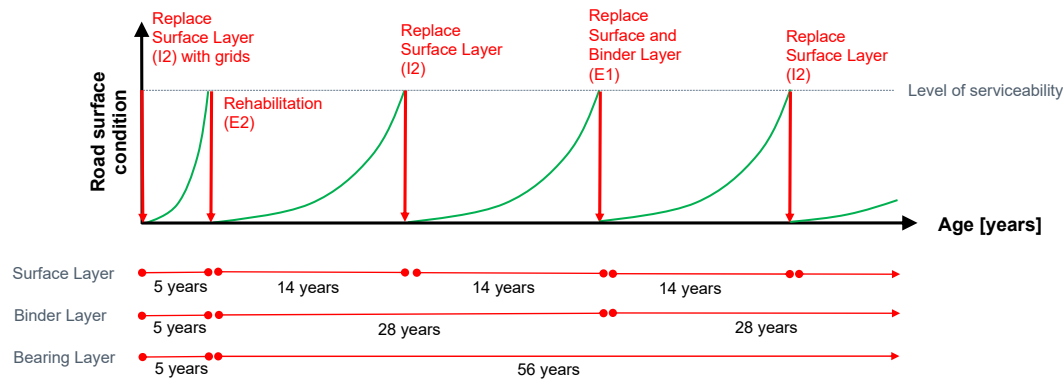


Figure 131: Sequence of Measures for variant 2 (asphalt reinforcement with bad paving quality)

The three variants are presented and discussed below based on the performance over time of the individual road surface indicators (PI), the composite KPI and the associated economic costs.

8.9.2 Reliability

For the evaluation of the reliability of the three variants, the modelled performance curves for the indices I0, I5 and for the KPI reliability are shown below as examples. Figure 132 shows the performance of the index I0 for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

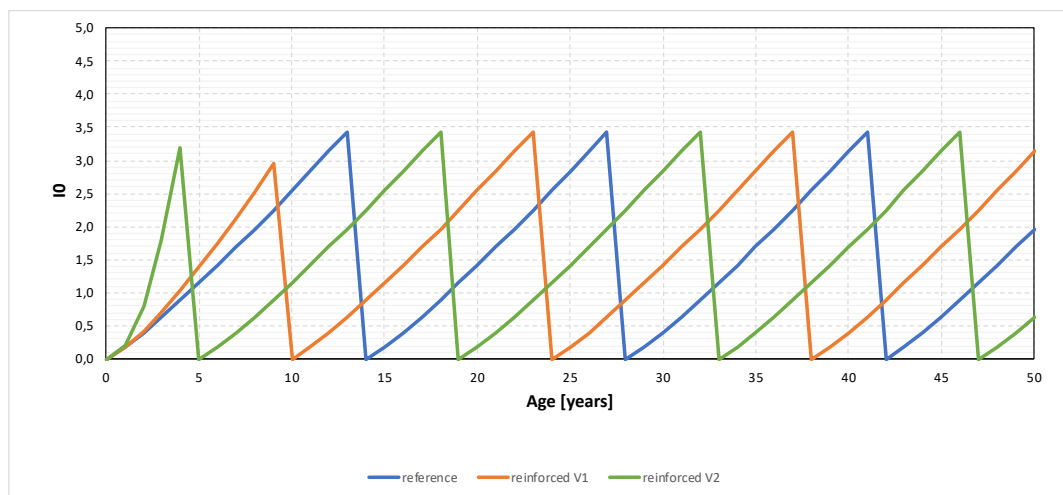


Figure 132: Performance Curves of Index I0

Figure 133 shows the progression of the index I5 over time for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

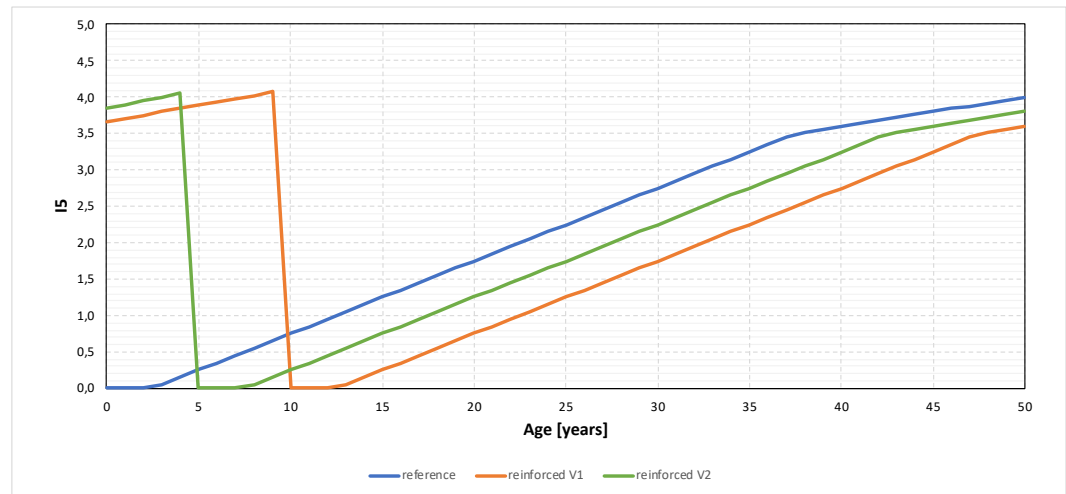


Figure 133: Performance Curves of Index I5

Figure 134 shows the progression of the KPI Reliability over time for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

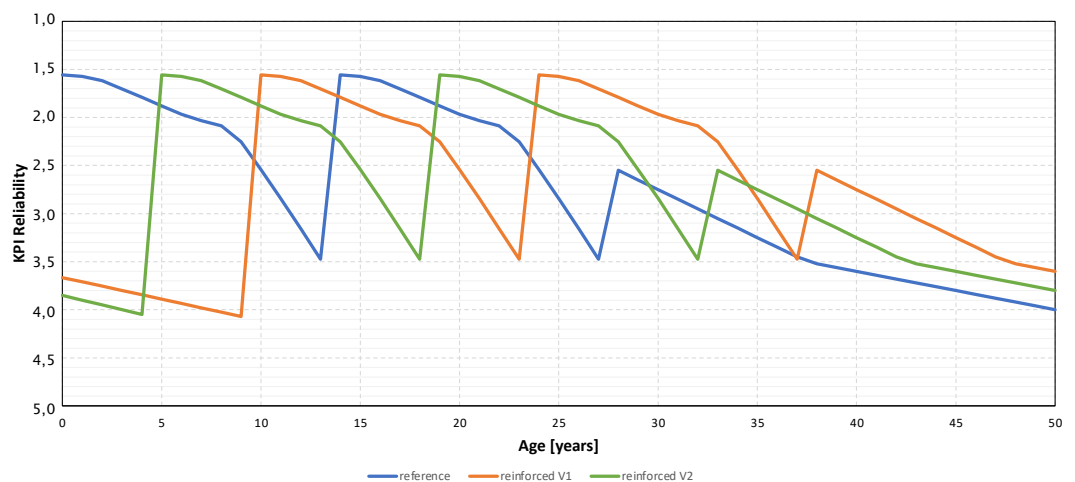


Figure 134: Performance Curves of KPI Reliability

To quantify the benefit, the area under the performance curve is determined for all three variants. For this purpose, the integral of the respective KPI curves (area under the curve (AUC)) is determined over the analysis period of 50 years. The results are summarised in Table 24.

Quantification of benefits based on the reliability KPI for the three model variants

	Reference	V1	V2
AUC	163,0	163,3	163,9

Table 24: Quantification of benefits based on the reliability KPI for the three model variants

8.9.3 User safety

For the evaluation of the user safety of the three variants, the state progressions for the indices I2, I3, I4 and for the KPI User Safety are shown below.

Figure 135 shows the progression of index I2 over time for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

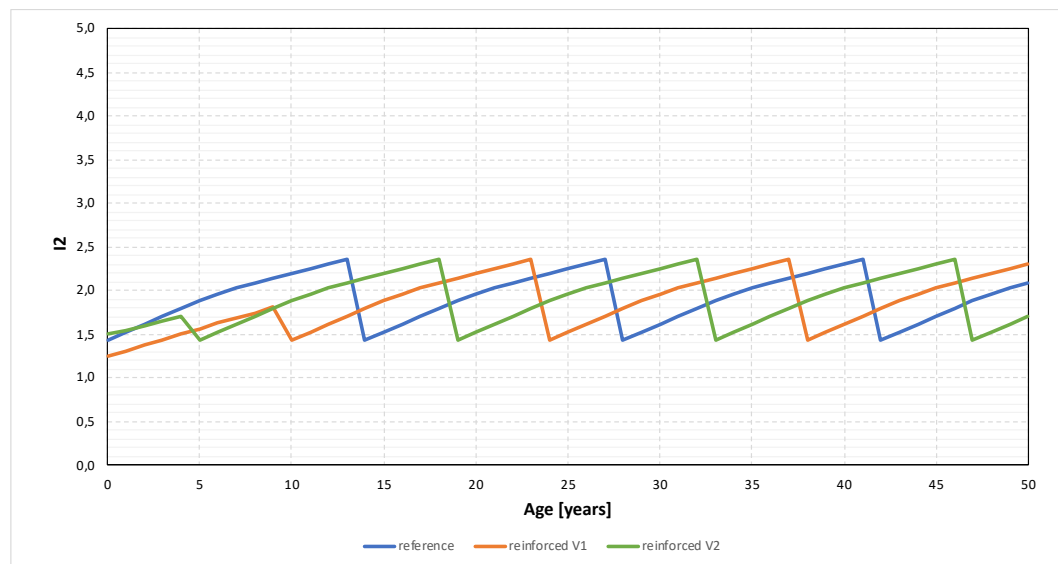


Figure 135: Performance Curves of Index I2

Figure 136 shows the progression of rutting over time for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

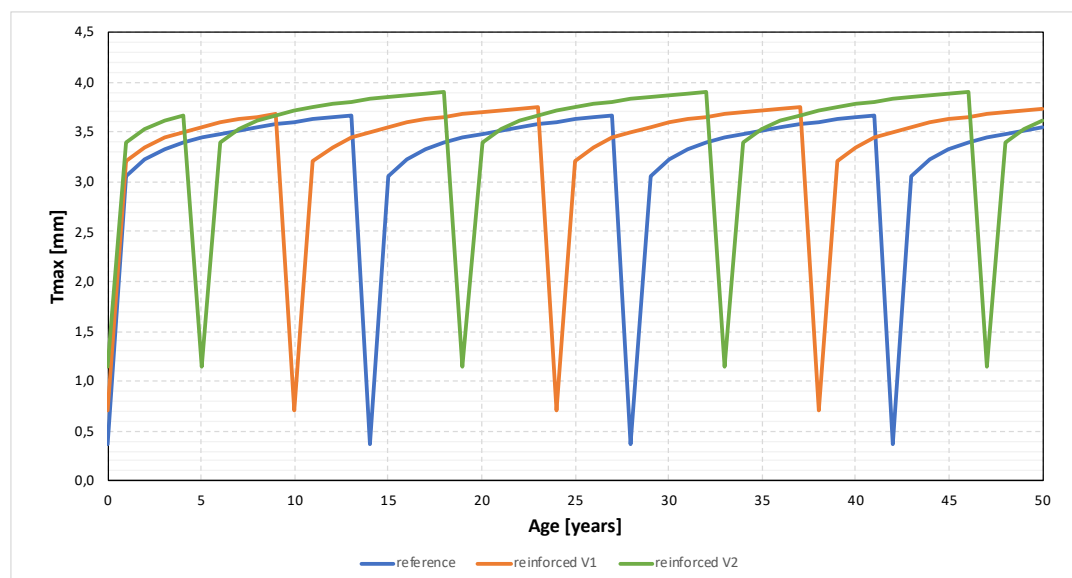


Figure 136: Performance Curves of rutting (Tmax)

Figure 137 shows the progression of skid resistance (μ SKM) over time for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

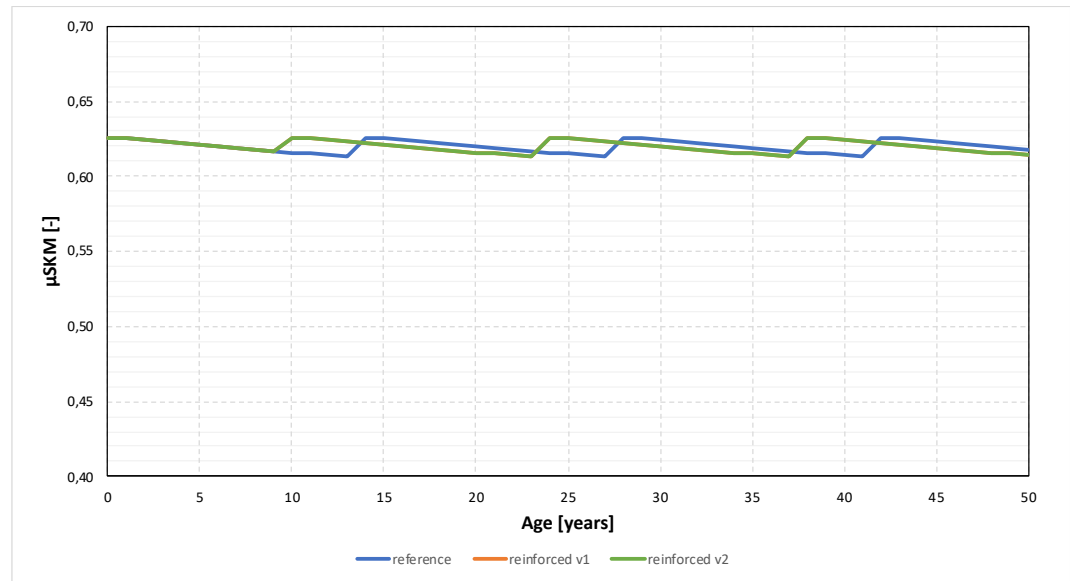


Figure 137: Performance Curves of skid resistance

Figure 138 shows the progression of the KPI User Safety over time for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

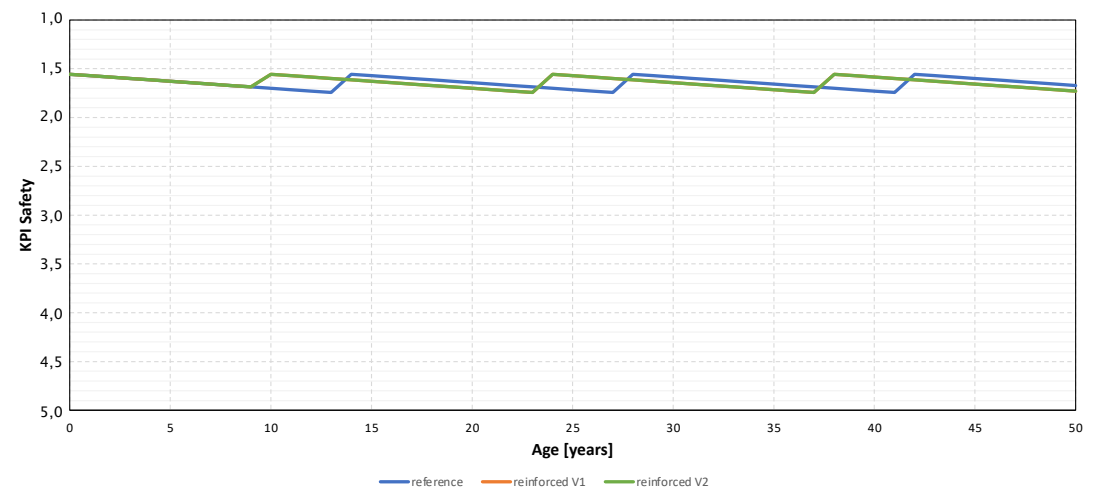


Figure 138: Performance Curves of KPI User Safety

To quantify the benefit, the area under the performance curve is determined for all three variants. For this purpose, the integral of the respective KPI curves (area under the curve (AUC)) is determined over the analysis period of 50 years. The results are summarised in Table 25.

Quantification of benefits based on the user safety KPI for the three model variants

	Reference	V1	V2
AUC	217,8	217,9	217,9

Table 25: Quantification of benefits based on the user safety KPI for the three model variants

8.9.4 Availability

To assess availability, the traffic impact of the maintenance measures within the analysis period must be estimated. For this purpose, the additional journey times generated by congestion and diversion routes are calculated and added up for each variant. As a traffic simulation was not available for this project, a simplified calculation approach was used. This contains the following assumptions for the route section under consideration:

Assumptions to calculate the KPI availability

Description	Parameter	Value
Additional journey time per vehicle [min]	–	3
Average Annual Daily Traffic [veh/24h]	AADT	12.000
Average Annual Daily High Traffic [veh/24h]	AADHT	1.500

Table 26: Assumptions to calculate the KPI availability

Based on the assumptions presented and measure-specific duration of the work (see Chapter 8.7), the following additional journey times are calculated for each type of measure.

Figure 139 shows additional travel times determined for reference variant and variants 1 and 2 with asphalt reinforcement.

Additional travel times per measure type

Measure type	I2	E1	E2
Additional travel time AADT [h]	4.200	8.400	12.600
Additional travel time AADHT [h]	525	1.050	1.575
Sum of additional travel times [h]	4.725	9.450	14.175

Table 27: Additional travel times per measure type

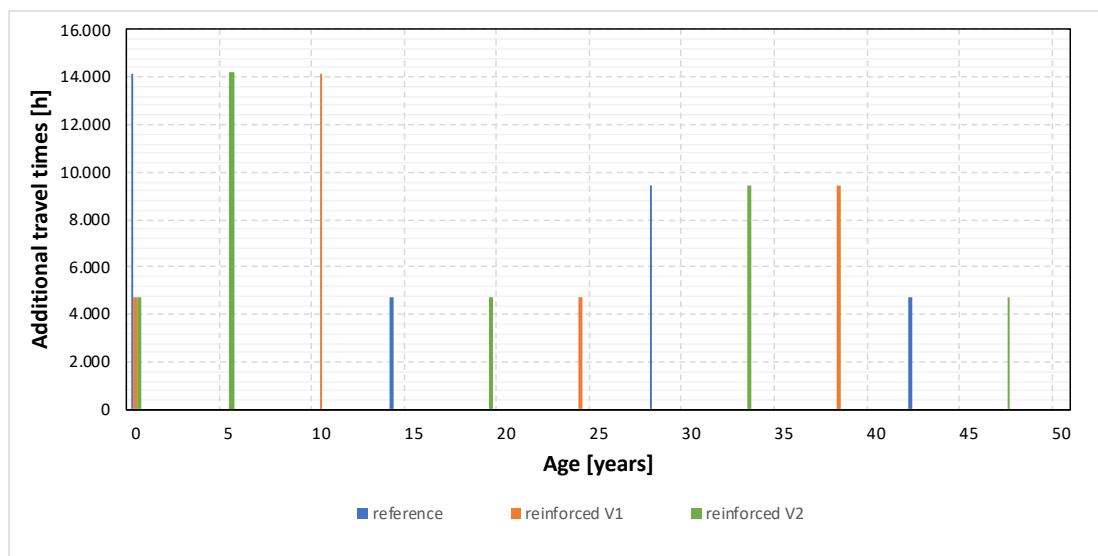


Figure 139: Additional travel times in dependence on the measure scenario

The normalization function shown in Figure 140: was used to convert additional travel times into the KPI availability.

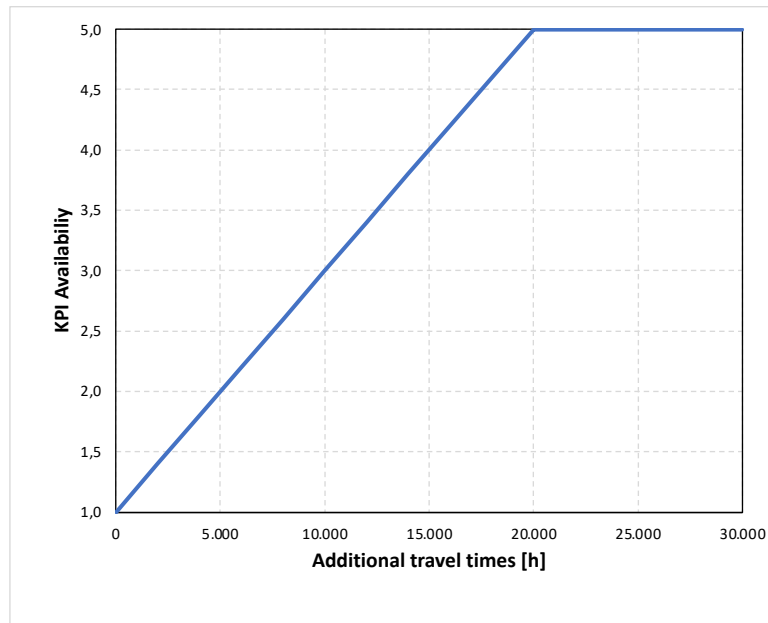


Figure 140: Normalization function to convert additional travel times into KPI availability

Figure 141 shows the KPI availability transformed from additional travel times for reference variant and variants 1 and 2 with asphalt reinforcement.

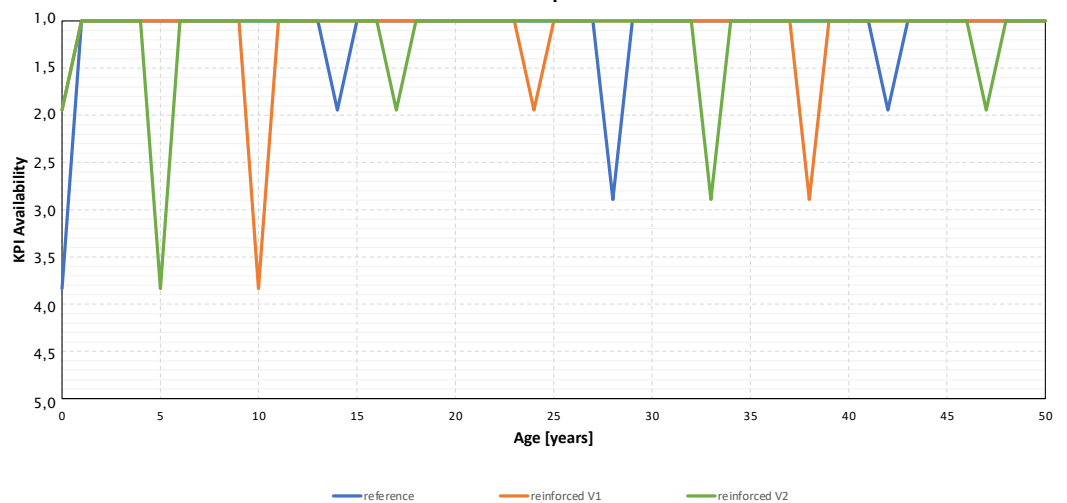


Figure 141: Performance Curves of KPI availability

To quantify benefit, area under performance curve is determined for all three variants. For this purpose, integral of the respective KPI curves (area under curve (AUC)) is determined over an analysis period of 50 years. The results are summarised in Table 28.

Quantification of benefits based on the KPI availability for the three model variants

	Reference	V1	V2
AUC	244,8	243,9	242,9

Table 28: Quantification of benefits based on the KPI availability for the three model variants

8.9.5 Sustainability

Sustainability is assessed depending on respective life cycle phase and type of measure. For this purpose, the life cycle phases of production, construction and maintenance and reuse/recycling were considered. Table 29 shows the equivalents of the CO₂, ADPE and ADPF indicators for the individual building materials in the production phase.

Input parameters for the production phase

Lifecycle phase	Material	CO ₂ eq.	ADPE eq.	ADPF eq.	Unit
A1-A4	AC T	188,548	0,03024	7.682.350	kg/m ³
A1-A4	AC B	206,276	0,03641	10.075.300	kg/m ³
A1-A4	AC D	202,937	0,03787	10.613.250	kg/m ³
A1-A4	Glas grid 50 kN	0,00049	1,5207E-07	7,46	kg/m ²
A1-A4	Glas grid 100 kN	0,00098	3,0413E-07	14,92	kg/m ²
A1-A4	Combi spray	0,35406	2,2439E-07	7,47	kg/m ²
A1-A4	Carbon grid	0,0056	–	67,14	kg/m ²
A1-A4	Synthetic grid	0,00038	7,2514E-11	0,01103	kg/m ²
A1-A4	Nonwoven	0,35308	7,2324E-08	0,01212	kg/m ²
A1-A4	Emulsion	0,166	0,00201	37.154	kg/LKW km

Table 29: Input parameters for the production phase (Mollenhauer et al. 2023)

Table 30 shows the equivalents of the indicators CO₂, ADPE and ADPF for individual building materials and processes (cleaning and transport) in construction and maintenance phase.

Input parameters for the construction and operational phase

Lifecycle phase	Material	CO ₂ eq.	ADPE eq.	ADPF eq.	Unit
A5	AC T	1,700	0,00015	23,40	kg/m ³
A5	AC B	1,918	0,00017	26,40	kg/m ³
A5	AC D	2,223	0,00019	30,60	kg/m ³
A5	Cleaning	1,749	0,00015	23.478	kg/m ²
A4	Transport Asphalt	1,345	0,00012	18.060	kg/LKW km
A4	Transport Grid	1,345	0,00012	18.060	kg/LKW km
A4	Transport Emulsion	2,287	0,00020	30.702	kg/LKW km

Table 30: Input parameters for the construction and operational phase

Table 31 shows the equivalents of indicators CO₂, ADPE and ADPF for individual building materials and processes (removal, cleaning, landfilling, recycling, transport) as part of reuse phase and recycling.

Input parameters for the recycling phase					
Lifecycle phase	Material	CO ₂ eq.	ADPE eq.	ADPF eq.	Unit
C4	AC T	-69,722	-0,0197	-5.114,50	kg/m ³
C4	AC B	-69,722	-0,0197	-5.114,50	kg/m ³
C4	AC D	-69,722	-0,0197	-5.114,50	kg/m ³
C3	dumping	36,146	0,00366	512,80	kg/m ³
C4	preparation	7,057	0,00001	134,00	kg/m ³
C1	Glas grid 50 kN	0,00000327	3,32E-10	0,00	kg/m ²
C1	Glas grid 100 kN	0,00000655	6,64E-10	0,00	kg/m ²
C1	Combi spray	0,0527	-0,00000579	0,00	kg/m ²
C1	Carbon grid		–	–	–
C1	Synthetic grid		–	–	–
C1	Nonwoven	0,0527	-0,00000583	0,00	kg/m ²
C1	Emulsion	12,2	–	–	kg/m ²
C1	Removal AC T 14 cm	7,764	–	–	kg/m ²
C1	Removal AC B 8 cm	4,436	–	–	kg/m ²
C1	Removal AC D 4 cm	2,218	–	–	kg/m ²
C1	Cleaning	1,749	0,00015	23.478	kg/m ²
C2	Transport plant	1,345	0,00012	18.060	kg/LKW km
C2	Transport dump	1,345	0,00012	18.060	kg/LKW km

Table 31: Input parameters for the recycling phase (Mollenhauer et al. 2023)

After multiplying by the corresponding volume or mass proportions of the construction materials and layers and adding up the values, the CO₂ equivalents shown in Table 32 result for the three types of measures analysed, in each case with and without the use of asphalt reinforcement.

Calculated CO ₂ equivalents in kilo tonnes for measure types I2, E1 and E2			
Measure type	I2	E1	E2
reference	55	108	167
reinforced	56	109	167

Table 32: Calculated CO₂ equivalents in kilo tonnes for measure types I2, E1 and E2

Figure 142 shows the CO₂ equivalents determined for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

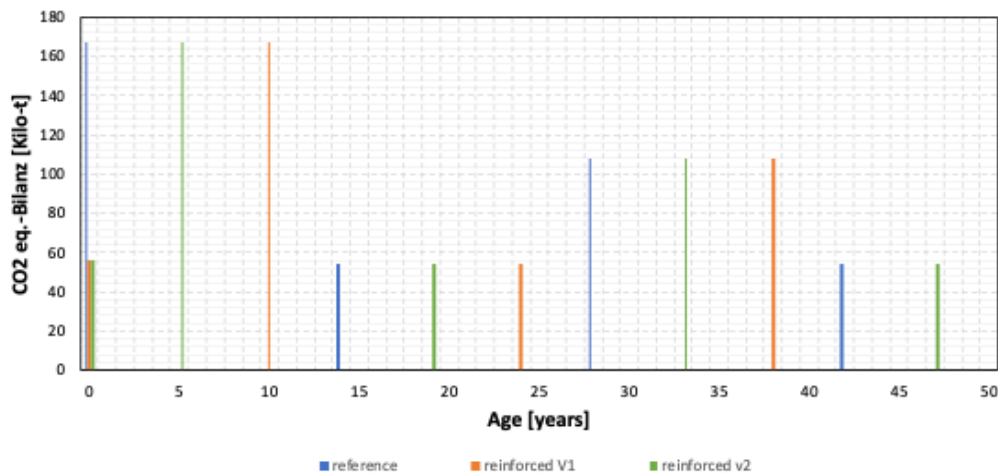


Figure 142: CO₂ equivalents determined for the reference variant and the two variants 1 and 2 with asphalt reinforcement

The normalization function shown in Figure 143 was used to convert the CO₂ equivalents into the KPI sustainability.

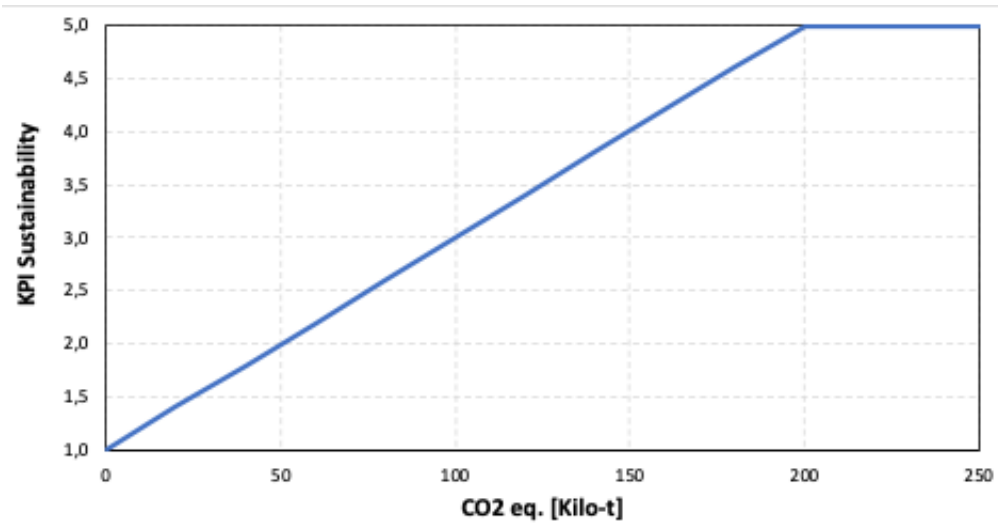


Figure 143: Normalization function to convert additional travel times into KPI sustainability

Figure 144 shows the KPI sustainability transformed from the CO₂ equivalents for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

To quantify the benefit, the area under the performance curve is determined for all three variants. For this purpose, the integral of the respective KPI curves (area under the curve (AUC)) is determined over the analysis period of 50 years. The results are summarised in Table 33.

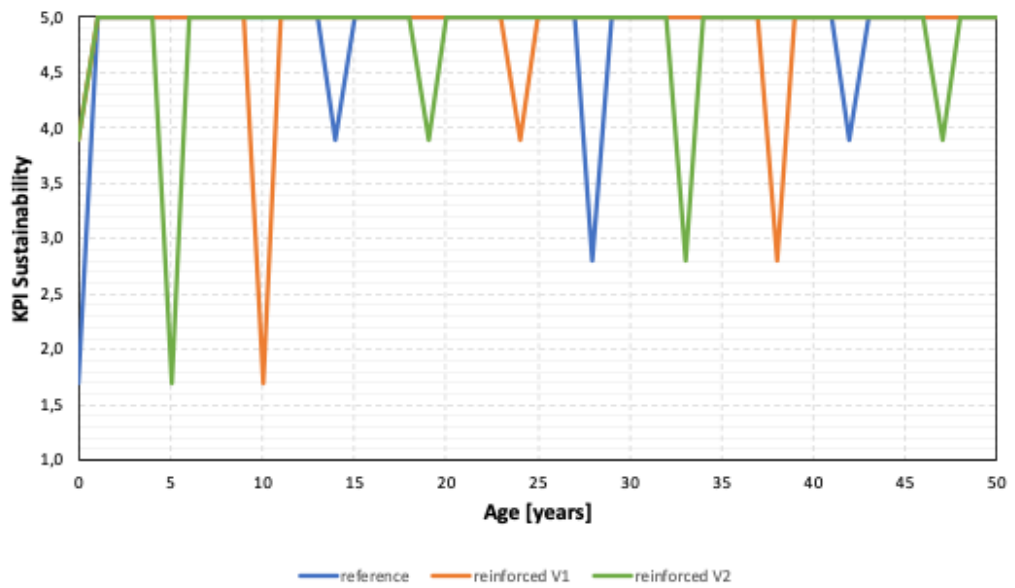


Figure 144: Performance Curves of KPI Sustainability

Quantification of benefits based on the KPI sustainability for the three model variants

	Reference	V1	V2
AUC	244,0	242,9	241,8

Table 33: Quantification of benefits based on the KPI sustainability for the three model variants

8.9.6 Costs

As explained in chapter 8.4.5, only the direct process or measure-related costs are considered for this study. External costs (e.g. additional travel times, CO₂ equivalents) are not monetised.

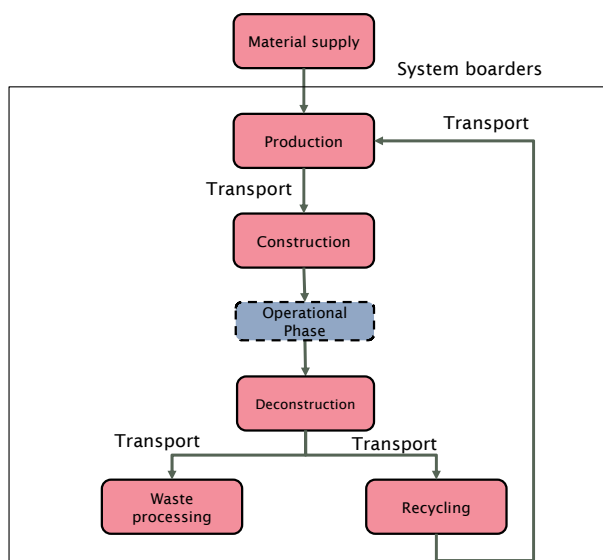


Figure 145: Process steps for analyzing costs

The costs are determined depending on the respective life cycle phase and the type of measure. The life cycle phases of production, construction and maintenance and reuse/recycling were considered. Table 34 shows the costs determined for the individual building materials in the production phase.

Costs of construction materials during the production phase

Lifecycle phase	Material	Costs (BPI 2020 incl. VAT)	Unit
A1-A4	AC T	119,70	CHF/m ³
A1-A4	AC B	141,70	CHF/m ³
A1-A4	AC D	217,40	CHF/m ³
A1-A4	Glass grid 100 kN	2,30	CHF/m ²
A1-A4	Non-woven	0,95	CHF/m ²
A1-A4	Emulsion	1,40	CHF/m ²

Table 34: Costs of construction materials during the production phase (Mollenhauer et al. 2023)

Table 35 shows the calculated costs for the individual building materials and processes (cleaning and transport) during the construction and maintenance phase.

Costs of construction materials during the construction and operational phase

Lifecycle phase	Material	Costs (BPI 2020 incl. VAT)	Unit
A5	AC T	177	CHF/m ³
A5	AC B	219	CHF/m ³
A5	AC D	308	CHF/m ³
A5	Glass grid 100 kN	1	CHF/m ²
A5	Non-woven	2,5	CHF/m ²
A5	Emulsion	0,012	CHF/m ²
A5	Cleaning	0,155	CHF/m ²

Table 35: Costs of construction materials during the construction and operational phase (Mollenhauer et al. 2023)

Table 36 shows the calculated costs for the individual building materials and processes (removal, cleaning, landfilling, recycling, transport) as part of the reuse phase or recycling.

Costs of construction materials during the recycling phase

Lifecycle phase	Material	Costs (BPI 2020 incl. VAT)	Unit
C3	Deposit	50	CHF/m ³
C4	Preparation	22	CHF/m ³
C1	Removal AC T 14 cm	2,99	CHF/m ²
C1	Removal AC B 8 cm	1,56	CHF/m ²
C1	Removal AC D 4 cm	0,8	CHF/m ²
C2	Transport to dump	2,23	CHF/km

Table 36: Costs of construction materials during the recycling phase (Mollenhauer et al. 2023)

After multiplying by the corresponding volume or mass proportions of the construction materials and layers and adding up the values, the costs shown in Table 37 result for the three types of measures analysed, both with and without the use of asphalt reinforcement.

Calculated costs of the measure types I2, E1 und E2

Measure type	I2	E1	E2
reference	60.296	113.990	224.376
reinforced	75.483	129.177	238.058

Table 37: Calculated costs of the measure types I2, E1 und E2

The total costs include the measure costs over a period of 50 years for the analysed section of road. These costs are to be updated to a predetermined point in time. The annual discount rate used for this purpose was set at 2 %. In principle, the discount rate can be set by the decision-maker. shows the cumulative costs (see Table 38) for the reference variant and the two variants 1 and 2 with asphalt reinforcement.

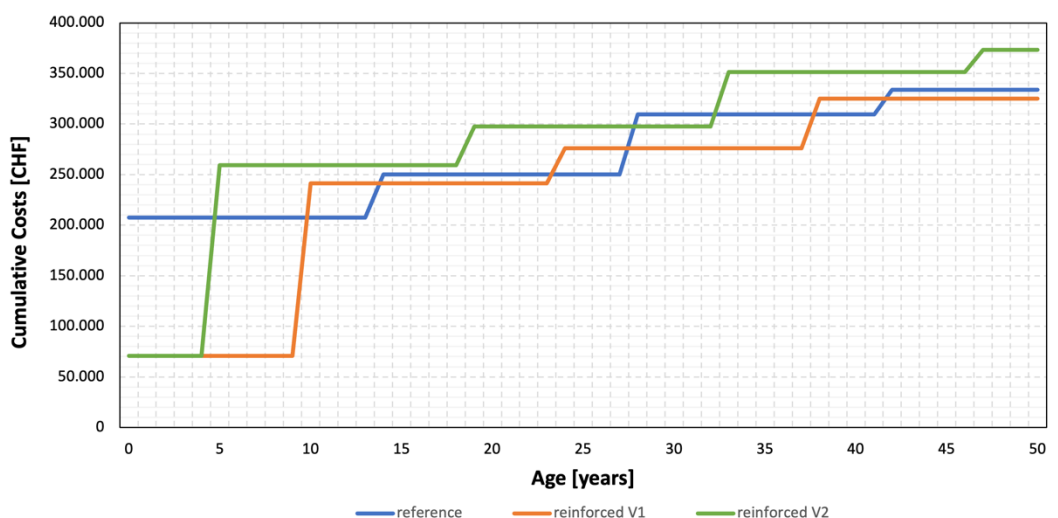


Figure 146: Cumulative process-related costs for the analysed variants including a discount rate of 2 %

Cumulative process-related measure costs for the three variants

	Reference	V1	V2
Cumulative process-related costs [CHF]	360.371	348.844	401.230

Table 38: Cumulative process-related measure costs for the three variants

As part of a life cycle approach, the "residual value" of a road section under consideration must also be determined at the end of the analysis period. At the end of the analysis period, the three model variants have different KPI Reliability values. While variant 1 requires an I2 measure after a further 6 years, the reference variant requires rehabilitation (E2) after 10 years. Due to the larger number of measures within the analysis period, a further E2 measure is only necessary after 15 years for variant 2 (see Figure 147:).

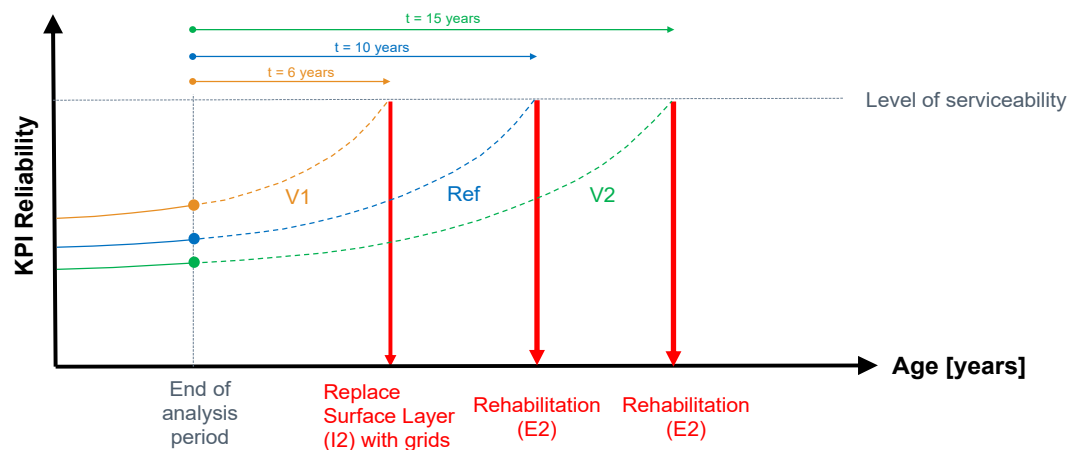


Figure 147: Calculation of residual value

The total costs are made up of the difference between the discounted measure costs (E_a) and the discounted residual value (R_n) (SN 940907):

$$\sum E_a \cdot q^{-(t-\tau)} - R_n \cdot q^{-(t-\tau)} \quad (13)$$

Table 39 shows the total costs of the three variants based on the discounted measure costs and residual costs.

Total costs of the three variants

	Reference	V1	V2
Discounted measure costs [CHF]	333.958	325.020	373.459
Discounted residual costs [CHF]	168.286	177.055	164.772
Total costs [CHF]	165.672	147.965	208.687

Table 39: Total costs of the three variants

8.10 Summary

The initial situation of this use case was that an existing asphalt road had cracks in the surface course. The lower asphalt layers have also already used up most of their intended service life. The question now arose as to whether a thorough renewal or replacement of all asphalt layers or to replace only the asphalt surface layer in combination with the use of asphalt reinforcement.

Three variants were compared, and their performance compared using defined key performance indicators (KPIs). The reference variant provides for the replacement of all asphalt layers at the beginning of the analysis period. Variants 1 and 2 consist of replacing the asphalt surface layer in combination with asphalt reinforcement between surface and binder layer. As a result, the time for replacing all three asphalt layers can be postponed. As the achieved pavement quality plays a central role in the use of asphalt reinforcement, two variants were considered. Variant 1 assumes that optimum conditions prevailed during paving, leading to the best possible paving quality. This means that the layer bond between the grid and asphalt layers could be optimally achieved. Variant 2, on the other hand, assumes that an optimum bond between the asphalt grid and the asphalt layers could not be achieved. As a rule, this means that the traffic loads applied to the asphalt surface courses cannot be optimally transferred to the underlying layers and the asphalt surface layer fatigues prematurely.

Table 40 summarises the results of the study for an analysis period of 50 years. It shows the benefits, which are mapped using the defined KPIs and quantified using the sum of the areas under the KPI curves. The corresponding costs include the purely measure-related costs.

Summary of results of the analysed model variants			
Model variant	Reference	V1	V2
Benefit cumulative [AUC]	869,5	867,9	866,6
Total costs [CHF]	165.672	147.965	208.687
Benefit/cost-ratio	5,2	5,8	4,2

Table 40: Summary of results of the analysed model variants

The three model variants show similar benefits in terms of the performance of the four KPIs. The reference variant Ref achieved a total area of 869.5 units, Variant V1 a total area of 867.9 units, and Variant V2 a total area of 866.6 units. However, there is a significant difference between the three model variants in terms of total costs. Variant V2, with total costs of CHF 208,687, is the least favourable option, as it requires an additional full reconstruction (E2) within the analysis period. In contrast, the reference variant Ref with CHF 165,672 and variant V1 with CHF 147,965 delivered lower costs. The results of this use case show that the use of asphalt reinforcement can provide an additional benefit regarding a long-term analysis period of 50 years. If the asphalt reinforcement is not installed optimally or cannot fulfil the minimum requirements, such as layer bonding, an additional maintenance measure may be necessary, leading

to increased costs within the life cycle. However, the use of asphalt reinforcement can have an advantage where the technically appropriate replacement of all asphalt layers is temporarily not feasible due to traffic, economic, personnel or other reasons. In addition, the impact of the necessary dumping of asphalt mill material contaminated with reinforcement mesh was investigated. Depending on the product-specific Environmental Product Declarations (EPDs) used to assess CO₂ equivalent units and other sustainability indicators, this can lead to further decrease in benefit.

8.10.1 Additional scenario disposal (of asphalt surface course)

When removing asphalt reinforcement, the reclaimed asphalt from the affected layers is rarely recycled in practice. Most of the time, mixing plants refuse this asphalt for recycling and instead it is dumped or thermally treated. This results in a poorer environmental and economic performance.

In the previous calculations, it was assumed that all layers (including the reinforced asphalt surface course) would be recycled if all bound layers were replaced.

The following scenario considers all bound layers to be replaced after the period of applied asphalt reinforcement and reinforced asphalt surface course as dumped. This rehabilitation results in higher costs (+CHF 2,520) and additional CO₂ equivalent of 15 tonnes (see Table 41).

Comparison of E2 measure with/without recycling of asphalt surface layer

Model variant	recycling	disposal
CO ₂ -equivalent [tonnes]	167	182
Measure costs [CHF]	207.726	210.246

Table 41: Comparison of E2 measure with/without recycling of asphalt surface layer

Regarding the overall evaluation of scenario 1, this leads to a marginal increase in total costs of 1.7%. In terms of CO₂ equivalents, this means an increase of 15 tonnes from 385 tonnes of CO₂ equivalents to 400 tonnes and leads to an increase of 3.9% over the entire life cycle. The result of scenario disposal has also impact on the assessment of the sustainability KPI. While a CO₂ equivalent of 167 t leads to a score of 4.3 for the recycling variant due to the normalization function, the CO₂ equivalent of 182 t is given a score of 4.6 for the landfill variant. In terms of the increasing relevance of the topic of sustainability, however, it should generally be questioned whether it makes sense to use products that are intended for disposal and cannot be returned to the materials cycle. A higher weighting of CO₂ equivalents can be achieved by adjusting the normalization function (see Figure 139) in the future.

9 Project conclusions

In this project, several laboratory and field tests were carried out in collaboration between BFH and Empa to compare the performance of different reinforcement systems to the unreinforced situation.

For the laboratory investigation at BFH all the specimens were prepared following a grid installation protocol after a demonstration carried out by each grid supplier. At Empa, due to the size of the specimens and the complexity of the installation, the grid suppliers were invited to carry out the installation of their own grids. The installation of the grids on the test sections for the full-scale analysis was also carried out by each company in consultation with all parties involved in the construction. The test section with product A, the installation was conducted early in the morning on already cured tack coat, while other systems (e.g. system B and D) were installed on a relatively uncured tag coat, which therefore affected the bonding quality. This is especially true for System B which uses a very high tack coat rate.

In addition to the laboratory and in-situ investigation, an important modelling effort was carried out using the Finite Element Method with the aim of obtaining the most accurate field of stresses and strains for different reinforcement grids. This effort comprised initially, the simulation of relatively simple laboratory tests such as the Leutner test, in order to obtain parameters describing the bonding between layers. Using these inputs, the objective was to model the more complex MMLS3 loading test by developing a plug-in for the software Abaqus, that allows modelling different test situations in a simple and user-friendly way.

Finally, a life cycle cost analysis was carried out to illustrate the benefits and associated costs of using asphalt reinforcement compared to conventional construction and rehabilitation methods.

9.1 Laboratory investigations

The tests in the laboratory comprised besides the analysis of the material used to prepare the specimens an investigation of the interlayer bond properties of the different reinforcement systems in comparison to the unreinforced situation. Specifically, the test used to assess the interlayer bonding were the Leutner test, carried out at Empa, and the AST test, carried out at BFH. Additionally, the capability of different reinforcement grids to delay reflective cracking was evaluated using cyclic 4-point bending tests on small specimens at BFH. Further, using the same test method, the influence of grid overlapping was studied. In order to link the laboratory tests with the in-situ situation, a crack propagation study was carried out at Empa using the traffic simulator MMLS3 on medium sized specimens.

Regarding the outcome of the laboratory tests the following conclusions can be drawn:

- The evaluation of the interlayer bonding using the Leutner test showed that all reinforcement systems, except for System A, have a negative influence on the interlayer bonding, although there are systems that under laboratory conditions can achieve the Swiss standard (SN-EN-12697-48) requirement of 15 kN. Nevertheless, the maximum

shear forces of the investigated reinforcement systems differed quite significantly. Especially, System A achieved with an average of 33 kN even higher values than the unreinforced reference (average of 30 kN), followed by B with an average value of nearly 26 kN, while System C performed very poorly with an average maximum shear force of less than 6 kN. System D reached an average of 16 kN.

- The AST results equally showed that a reinforcement grid has a negative influence on the interlayer bonding. The effect of moisture damage showed that the strength decreases due to water conditioning of the specimens.
- A comparison between Leutner and AST test was not possible, since the achieved values totally depended on the construction and compaction of the specimens as well as the installation of the reinforcement. The obtained results showed that there existed huge differences between the Empa and the BFH specimens, visible from the lower AST values compared to the Leutner values without lateral force. On the other hand, a comparison of specimens of the unreinforced reference compacted at Empa between AST (without lateral load) and Leutner led to consistent results.
- The 4PB beam crack propagation test conducted at 5 °C revealed that all reinforced specimens sustained a higher number cyclic loading irrespective of the grid type in comparison to the unreinforced reference specimens. The specimens with grid reinforcement B offered the highest resistance against crack propagation followed by System D.
- The specimens with a chosen overlap length of 15 cm were found to be more resistant in the 4PB test, compared to the full-length specimens. This shows that overlapping of the tested reinforcement grids according to the suppliers' recommendations does not provide a problem and that the overlap length for a specific reinforcement product could even be optimized.
- The crack propagation tests conducted with the MMLS3 traffic load simulator showed that System A outperforms all others, while System D performs poorly, carrying only about 60 % of the loads compared to System A. According to deformation sensors (Lvdt) measurements System A provided high stiffness to the structure, whereas System D seems even weaker than the reference without reinforcement. This fact is extremely interesting since both systems consist of the same reinforcement material type including material stiffness and geometry.
- Using Digital Image Correlation (DIC) three distinct mechanisms of crack formation and propagation could be identified. Some specimens, immediately after loading shows debonding of the layers, followed by the initiation of a crack at the notch tip, which travels through the slab to the upper surface. In the second mechanism, a vertical crack starts at the notch, propagates directly to the interface causing layer debonding, and then continues to the top. This suggests that the interlayer is slightly stronger than in the first mechanism. In both cases, the vertical crack in the top layer can either continue from the crack in the bottom layer or be displaced horizontally. The horizontal displacement indicates a very weak interlayer, causing the layers to act independently under load. The third mechanism involves a vertical crack propagating directly to the top layer without causing interface separation. In the first two mechanisms, crack propagation generally occurred faster than in slabs where no interlayer separation was observed.
- Therefore, the type of failure is generally correlated with the specimen performance: if interlayer debonding occurs, the number of load cycles until failure is typically lower than if there is no interlayer debonding. In summary, the tests showed

that a System Can delay the crack formation and propagation if it provides a good bonding to the rest of the structure leading to a stiffening effect of the whole structure.

- In this way it was demonstrated that the combination of laboratory-scaled traffic simulator tests (MMLS tests), deflection measurements with Lvdts and digital image correlation (DIC) data analysis is an important tool to understand the mechanism of reinforcement systems, especially with the newly proposed way of determining horizontal and vertical crack propagation patterns. The evaluation of horizontal and vertical crack propagation demonstrates that the performance of reinforcements depends on different mechanisms and that it therefore important to not rely only on small-scale tests.
- When looking at the performed testing, as mentioned above, the MMLS3 crack propagation test must be considered more realistic and comparable to the field situation compared to the 4PB test due to its load configuration and geometry of the specimens, especially regarding the mesh size of the reinforcement products.
- Although the 4PB crack propagation test has its limitations, it is easier to perform and useful for comparing different products. To increase the confidence level of the test results, it is essential to conduct it using a larger number of specimens (not only 2 specimens).
- By using different laboratory test methods to determine the crack propagation (MMLS3 and 4PB), the crack-bridging and crack-retarding effect of asphalt reinforcement grids and the differences to systems without asphalt reinforcement could be clearly demonstrated for various asphalt reinforcement systems. The crack propagation test results showed that the reinforcement provided higher resistance against crack propagation. However, the effectiveness of the improvement depends on the type of reinforcement grid used and the interlayer bonding was found to be one of the controlling factors for the performance of a reinforced pavement system. To achieve better performance, it is therefore very important to improve interlayer bonding, which is also crucial for preventing moisture damage.

9.2 Test section investigations

The test track consisting of consecutive sections equipped with the reinforcement grid systems and a reference section without reinforcement grid was constructed in Sierre, Canton Valais. Before the construction and during the duration of the research project several surface survey campaigns were carried out monitoring the performance over time. In addition, cores were taken at regular time intervals and interlayer bonding was evaluated using the Leutner test.

The findings and the conclusions are the following:

- The interlayer bond test results of the field cores showed that the reference section has the highest value in comparison to the reinforced sections irrespective of the reinforcement type used and therefore the best interlayer bond properties. However, the product used in Section A has the higher values followed by Section B, while Sections C and D never seem to be able to reach the standard requirement of 15 kN.
- From the results it is also clear that the layer adhesion condition improved over time, a fact that is in accordance with earlier investigations (Raab et al., 2017). The

increase of shear force for systems A, B and the reference are in the order of 40 %, while the increase for System C and D is 66 % and 76 % respectively.

- Although System A could proof its superior bonding qualities in the laboratory, it is worth noting that its installation was conducted early in the morning on already cured tack coat, while other systems (e.g. system B and D) were installed on a relatively uncured tag coat, which therefore affected the bonding quality. This is especially true for System B which uses a very high tack coat rate. Further, as highlighted by the producer of a system with self-adhesive membrane, the required temperature of the surface layer of 160 °C, which they consider important to ensure the bonding properties of their system, was not achieved. Producers of self-adhesive systems also claim the time dependent mechanism of compacting traffic load in combination with high summer temperatures to guarantee the performance of their product.
- Comparing the field results with the laboratory Leutner test the following can be said: Although only 2 field cores were tested for each reinforcement system, the ranking appears to be similar (System A best, followed by System B). As visible in earlier investigations (Raab et al. 2017) there are systems that show in the laboratory a better or comparable behavior to unreinforced specimens, while in situ, due to construction constrains and difficulties, the reinforcement grids always provide a discontinuity between layers, often leading to lower interlayer bond properties. The poor performance of systems C and D was also found in the lab testing, while it is worth noting that System D tested on site (with chipping) was different to System D tested in the lab (without chipping).
- When looking at the test track performance in terms of reflective cracking, the reference section was found to have the highest number of reflective cracks (43 %) followed by Section B with 20 %. In Section A, the number of reflective cracks was 2 %, while for sections C and D no reflective cracks could be found.
- The field investigations therefore revealed that all installed reinforcements were quite capable of preventing or at least delaying the appearance of reflective cracks.
- However, it was difficult to compare the reinforcement grids installed in the different sections with each other, since the selected test track was part of the scheduled maintenance work of Canton Valais and was managed in collaboration with the Cantonal authorities. Neither the geometry (curves) nor the amount and regime of traffic (number of heavy trucks, turning lanes, shear forces due to breaking at roundabouts) was comparable for all sections. In this context, the roundabout presence at the beginning of Section A and turn-off lane in Section B must be mentioned. As visible in an inspection in April 2024, aggregate loss and ravelling of the surface layer were found, phenomena that cannot be attributed to the performance of the reinforcement but reveal the different loading stresses of the different sections as well as possible problems of the surface course mixture.

9.3 Modelling

- Within the scope of this research project, an accurate numerical simulation of the MMLS3 tests was developed and validated against experimental results, demonstrating high accuracy in predicting structural responses. The model incorporates advanced material properties, loading conditions, interlayer bonding behavior, and the ability to represent grid geometry and materials, including mesh size,

cross-sectional area, and mechanical properties. It effectively distinguishes the behavior of slabs with and without reinforcement. Future research should integrate specific grid properties to enable an individual evaluation of each reinforcement type.

- Continuing this modeling effort will facilitate the assessment of various reinforcement systems by utilizing product-specific data provided by suppliers. By simulating their performance in full-scale pavements, the model has the potential to significantly reduce the need for extensive physical testing while allowing for easy evaluation of different test conditions. Importantly, this approach provides critical input data, such as stress and strain responses, that are essential for mechanistic-empirical pavement design, enabling more accurate and efficient dimensioning of reinforced pavements.

10 Recommendations

- All evaluations and tests performed during the project revealed that the number of laboratory and field specimens must be increased to be able to provide reliable findings and conclusions. This is especially true regarding standardization processes and LCCA methods.
- The road owners and contractors must ensure that the necessary requirements for pavement construction are strictly met when installing reinforcement systems and that all conditions for an optimal installation of specific reinforcement grids are fulfilled. Additionally, a close collaboration between contractors and grid suppliers is essential.
- When looking at the performed crack evaluation in the laboratory, the MMLS3 crack propagation test must be considered more realistic and comparable to the field situation compared to the 4PB test (though its a small-scale test and easy to be perform) due to its load configuration and geometry of the specimens, especially regarding the mesh size of the reinforcement products. Therefore, it is recommended to conduct this type of medium-scaled test, whenever possible using deflection measurements with Lvdts and digital image correlation (DIC) data analysis to understand the mechanism of reinforcement systems, determining horizontal and vertical crack propagation patterns.
- To reduce the amount of testing and evaluate different test conditions, the modelling developed in the frame of this research project should be applied for determining the performance of different reinforcement systems. In this respect it is essential that the reinforcement suppliers provide all necessary input data.
In accordance with previous works, this research project demonstrated the ability of reinforcement grids to prevent reflective cracking in general, and it is therefore recommended to use such reinforcement systems for rehabilitation and maintenance measures.
- Since it was shown that there is a strong connection between the interlayer bonding properties and their ability to prevent reflective cracking on the one hand interlayer bond testing should be performed for benchmarking and ranking different systems, on the other hand suppliers of reinforcement grids need to optimize and/or improve their products in this respect.
- It would be interesting to conduct a survey, at least visual inspection and crack intensity measurements even after completion of the project to have more comprehensive analysis and outcome of the final conclusions.
- For field comparison of different reinforcement products, a test track with equal test sections in terms of traffic, alignment etc. would be desirable.

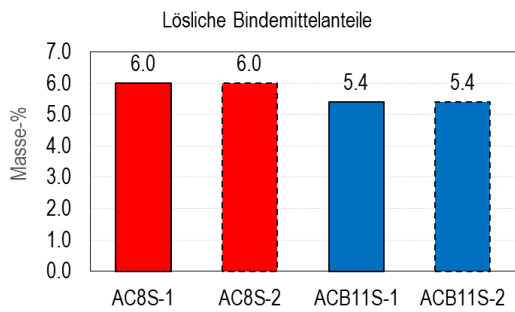


Figure A3: Bitumen content

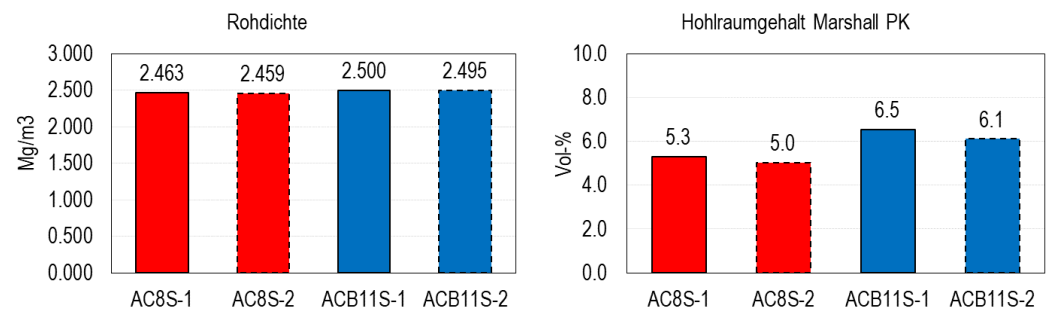


Figure A4: Max. density and void content

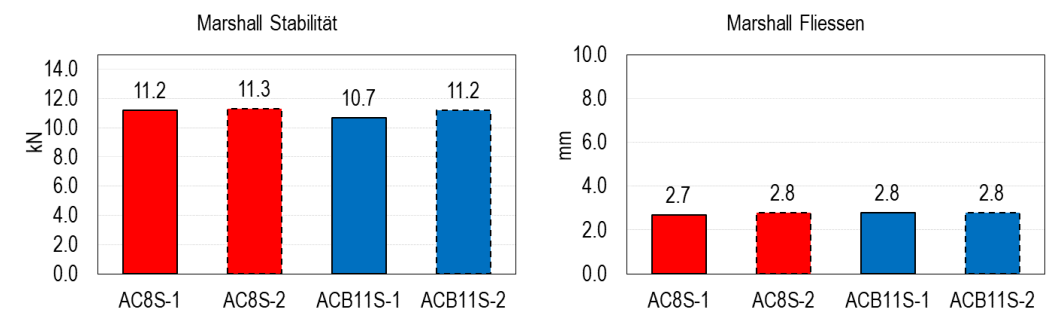


Figure A5: Marshal-Stability results

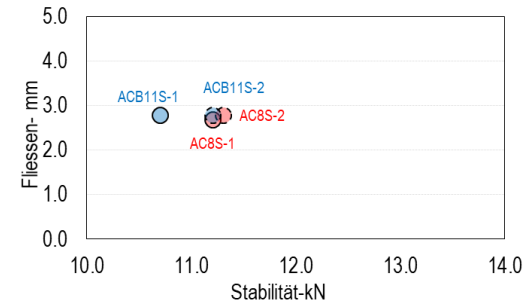


Figure A6: Marshal-Stability resultsa

11.1.3 Grading curves

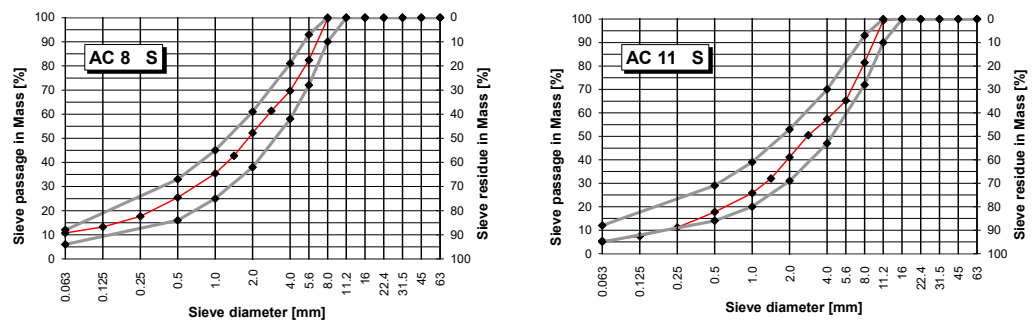


Figure A7: AC 8 S and AC 11 S grading curves

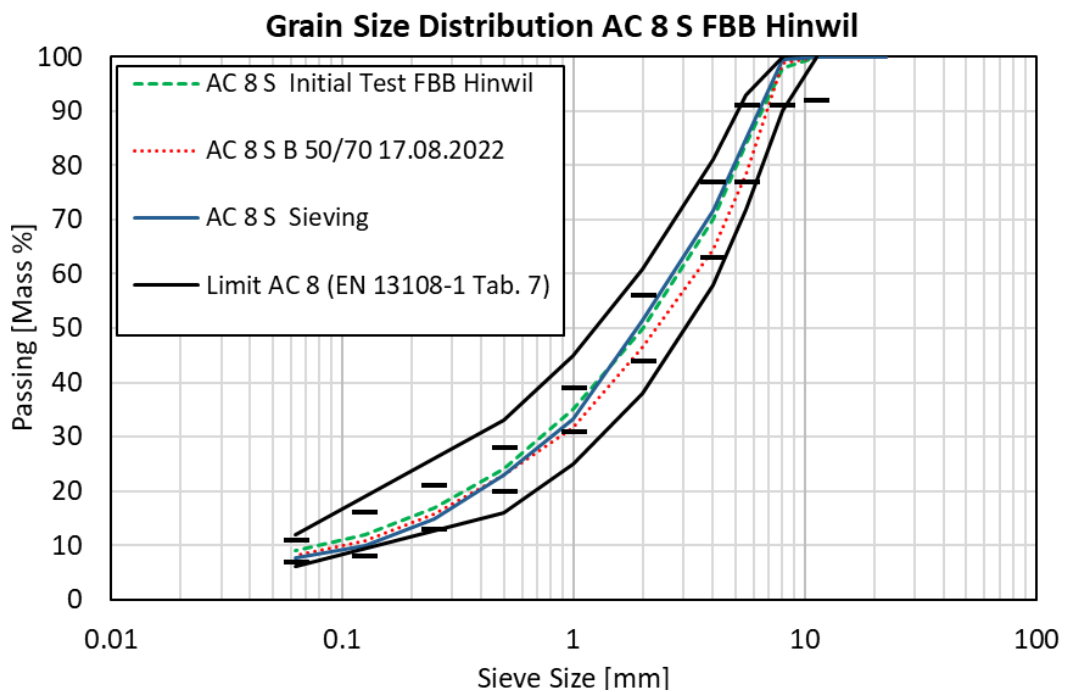
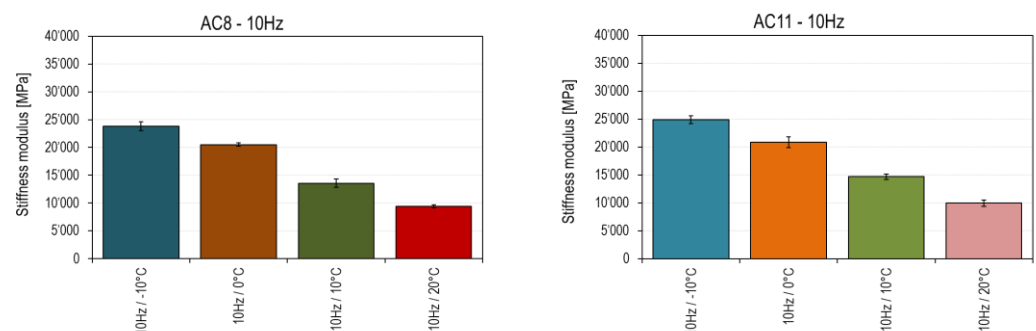


Figure A8: AC 8 S Mixture gradation

11.1.4 Stiffness (Marshall samples)



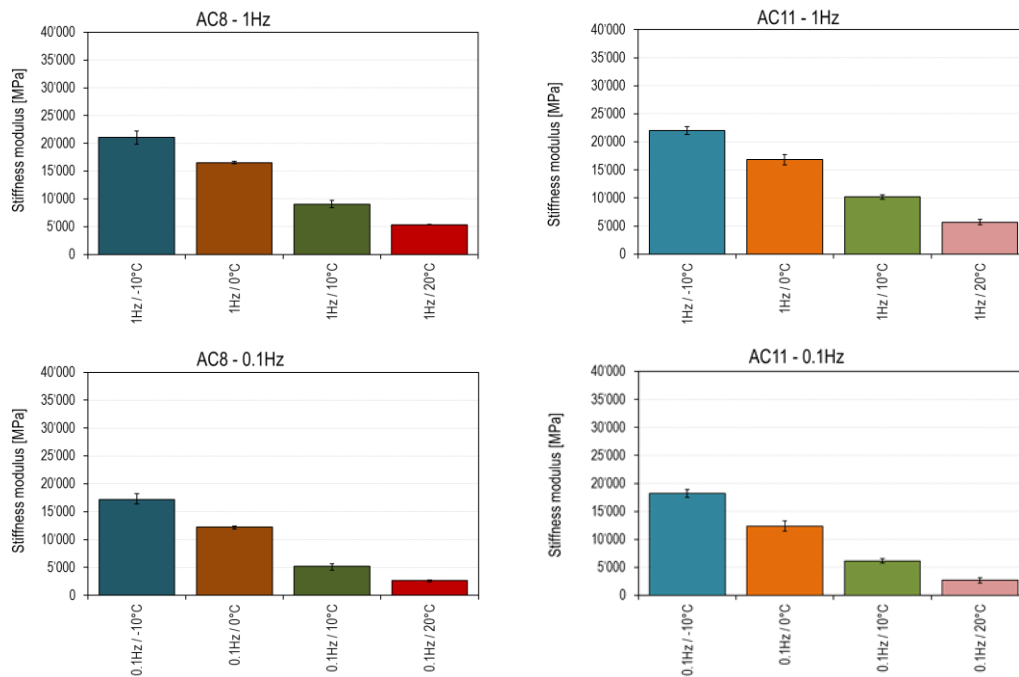


Figure A9: AC 8 S and AC 11 S Stiffness

11.1.5 Marshall stability

Mixture AC 8 S Marshall stability performance details

Name	d [mm]	h [mm]	m [g]	p [kg/m ³]	S [kN]	F [mm]	F T [mm]	F t [mm]	S/F [kN/mm]
1	101.5	63.1	1193.6	2337.8	9.2	2.4	2.5	1.6	3.8
2	101.5	63.2	1196.6	2340.0	9.7	2.7	2.7	1.6	3.6
3	101.5	63.2	1196.6	2340.0	9.7	2.5	2.5	1.6	3.8
4	101.5	63.1	1196.9	2344.3	9.7	2.6	2.6	1.6	3.8
Mittelwert	101.5	63.2	1195.9	2340.5	9.6	2.5	2.5	1.6	3.8
Std. Abw.			1.56	2.71	0.22				
d	= Durchmesser p			= Raumdichte			= Gesamtfließwert		
h	= Höhe			= Stabilität berichtigt			F T	= Fließwert tangential	
m	= Masse			= Fließwert			F t		
							S/F	= Marshall-Quotient	

Table A1: Mixture Marshall stability details of AC 8 S

Mixture AC 11 S Marshall stability performance details

Name	d [mm]	h [mm]	m [g]	p [kg/m ³]	S [kN]	F [mm]	F T [mm]	F t [mm]	S/F [kN/mm]
1	101.5	63.8	1195.6	2316.0	9.2	2.6	2.6	1.6	3.5
2	101.5	63.9	1195.7	2312.6	9.4	2.4	2.3	1.5	3.9
3	101.5	63.8	1196.3	2317.4	8.9	2.6	2.5	1.6	3.5
4	101.5	63.7	1193.9	2316.4	9.4	2.5	2.5	1.5	3.8
Mittelwert	101.5	63.8	1195.4	2315.6	9.2	2.5	2.5	1.5	3.7
Std. Abw.			1.03	2.08	0.23				
d	= Durchmesser p			= Raumdichte			= Gesamtfliesswert		
h	= Höhe S			= Stabilität berichtigt			F T	= Fliesswert tangential	
m	= Masse F			= Fliesswert			S/F	= Marshall-Quotient	

Table A2: Mixture Marshall stability details of AC 11 S

11.2 4PB beam specimen geometry

Beam specimen geometry of full-length grids

Sr. #	Specimen	Beam (mm)			Notch (mm)		Dry Wt. of Beam (g)
		Length	Width	Height	Depth	Width	
1	Ref-1	500.0	89.0	90.0	15	3.5	8642
2	Ref-2	499.0	85.8	87.0	14	3.5	8546
3	Cr-B-1	500.1	89.0	90.1	15	3.0	8860
4	Cr-B-2	500.0	89.0	88.2	16	3.1	8702
5	Cr-A-1	499.6	88.9	89.2	15	3.0	8815
6	Cr-A-2	500.1	89.1	88.0	14	3.1	8846
7	Cr-C-1	500.1	88.9	88.3	15	3.5	8778
8	Cr-C-2	500.2	89.1	88.2	15	3.8	8636
9	Cr-D-1	499.0	86.0	87.0	15	3.8	8946
10	Cr-D-2	500.0	89.0	88.0	15	3.5	8866

Table A3: Full-length gird beam specimen geometry

Beam specimen geometry of overlap length grids

Sr. #	Specimen	Beam (mm)			Notch (mm)		Dry Wt. of Beam (g)
		Length	Width	Height	Depth	Width	
1	Ref-1	500.0	89.0	90.0	15	3.5	8642
2	Ref-2	499.0	85.8	87.0	14	3.5	8546
3	Or-B-1	500.0	88.0	87.0	14	3.5	8483
4	Or-B-2	500.0	90.0	89.0	15	3.0	8678
5	Or-A-1	499.6	88.9	89.2	15	3.0	8815
6	Or-A-2	500.1	89.1	88.0	14	3.1	8846
7	Or-C-1	500.0	90.0	88.0	14	3.0	8621
8	Or-C-2	500.0	88.0	86.0	14	3.5	8581
9	Or-D-1	499.0	86.0	87.0	15	3.8	8946
10	Or-D-2	500.0	89.0	88.0	15	3.5	8866

Table A4: Overlap-length gird beam specimen geometry

11.3 Digital Image Correlation

The Digital Image Correlation (DIC) test setup was carried out using the following steps:

- place sample inside and close the window
- Place camera with approx. 30° angle. cameras should be one above the other
- Place the light above the camera (with greater angle)
- adjust the focus
 - put maximum light and open the aperture on the camera to the maximum
 - reduce the opening time of the camera on vic-snap (in order to have something visible)
 - adjust the focus on the camera to an optimum (minimum of noise -> maximum of purple)
 - put the camera aperture to a middle level (around 5.6) and lock it. -> adjust the brightness with the aperture time on vic-snap
- ready for calibration:
 - Don't move the camera or the light
 - Open the climate box in the middle.
 - Hold the calibrate plate 14mm (or bigger) in front of your sample (as close as possible, or remove your sample, if possible, to be in focus as well with the calibration plate).
 - adjust the light intensity or the camera aperture time on vic-snap to be slightly over-exposed on the dots of the panel (optimum, otherwise, it is also working, just less precise).
 - Take about 30 shots while tilting the calibration panel in all directions.
 - Calibration should be good (below 0.1).
 - Remove the calibration panel and reclose the climate box.

- Calibration optimisation on vic-3D
 - go to menu calibration -> calibrate stereo system
 - choose the imaging system "3 glass window" show in the checkbox on the bottom left corner (if not available, press edit, add one imaging system with the checkbox "complex distortion" checked and the option "VRO 5th order"). This allows recalculation while measuring through windows or liquids.
 - Recalculate your calibration. This should be lower than the previous one.
 - Control if the angle between the cameras is correct (calculated one and real one)

11.3.1 Time vs Force results of the reinforced specimens

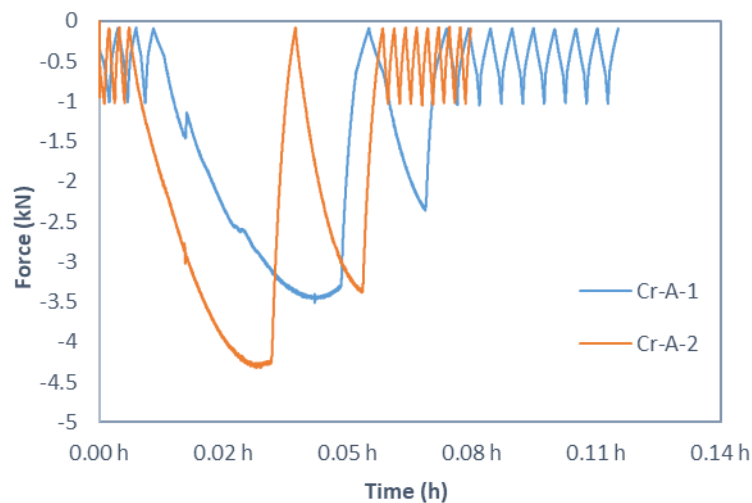


Figure A10: Force vs Time diagram for Cr-A Specimen

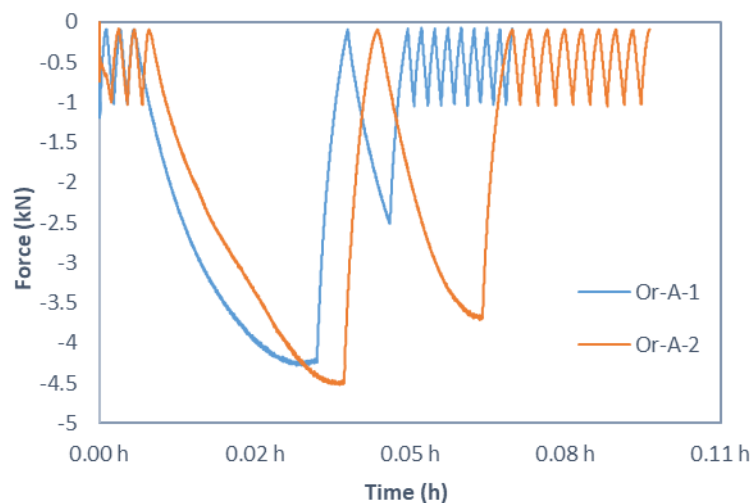


Figure A11: Force vs Time diagram for Or-A Specimen

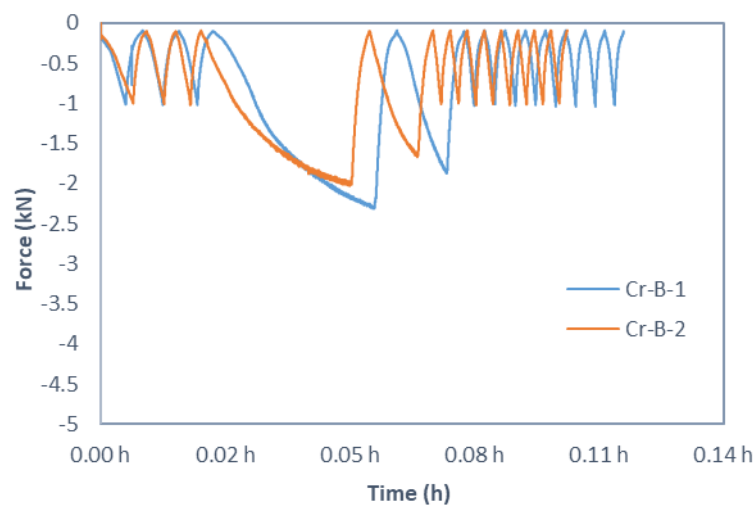


Figure A12: Force vs Time diagram for Cr-B Specimen

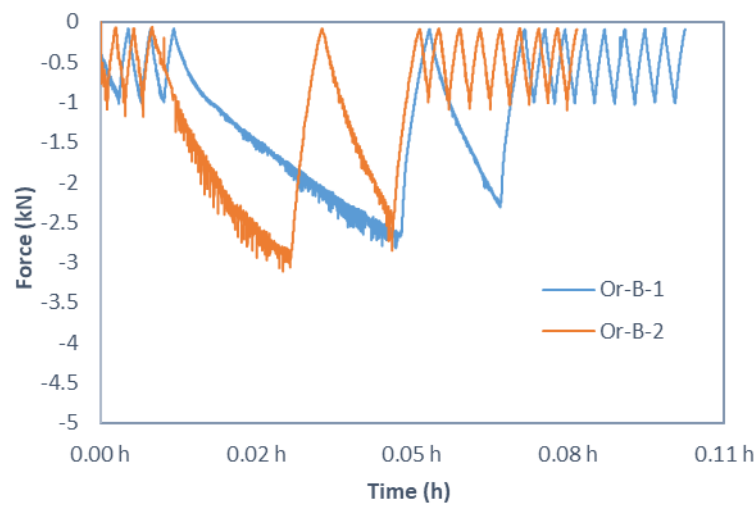


Figure A13: Force vs Time diagram for Or-B Specimen

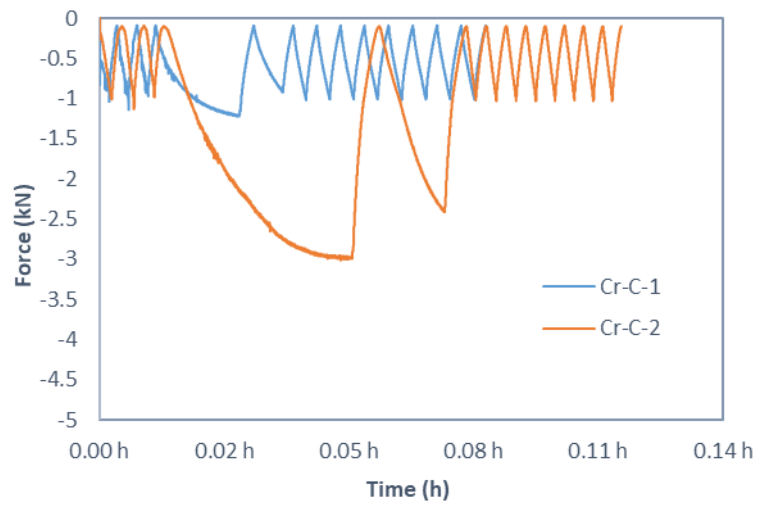


Figure A14: Force vs Time diagram for Cr-C Specimen

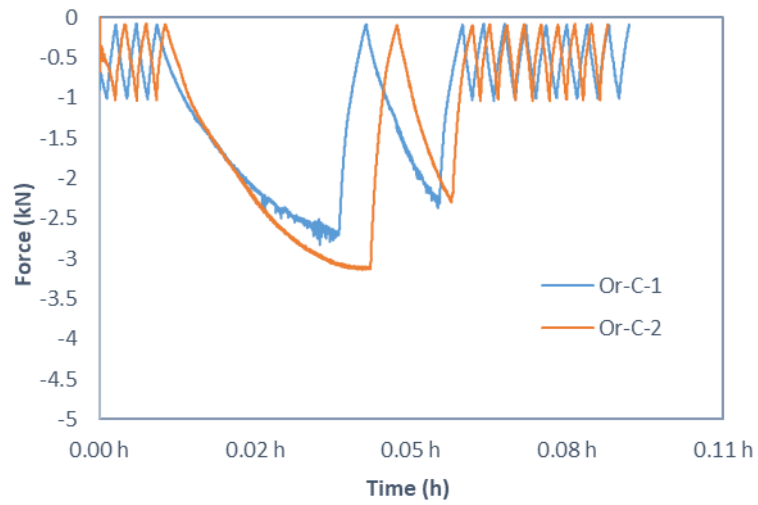


Figure A15: Force vs Time diagram for Or-C Specimen

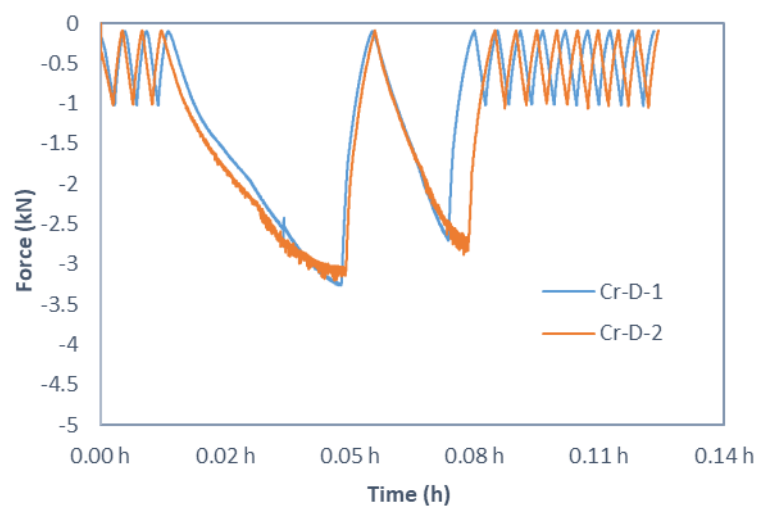


Figure A16: Force vs Time diagram for Cr-D Specimen

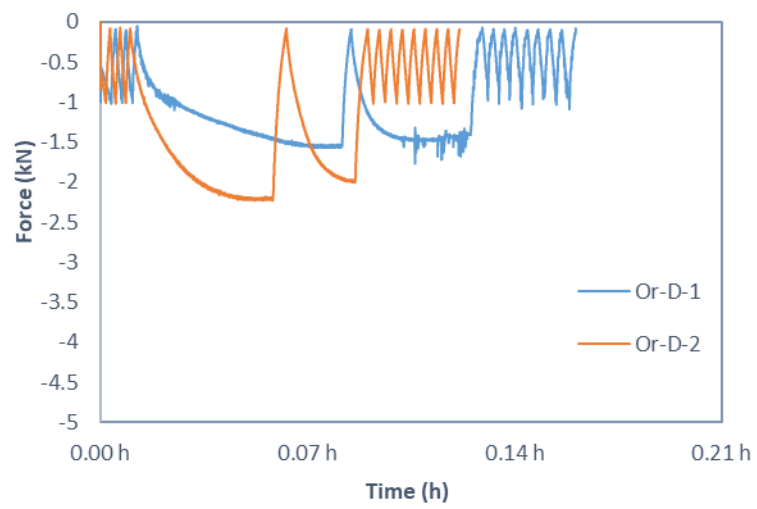


Figure A17: Force vs Time diagram for Or-D Specimen

11.4 Test Track data

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Test section Sierre - General information according to grid suppliers communications

- Test section concept: 40 mm milling / reinforcement grids / 40 mm AC 11 S (PmB Bitumen)

- Reinforcement grids on both lanes

Informations importantes/tâches pour entreprise mandatée par le canton du Valais
Tâches pour fournisseurs de grilles

Company	A	B	C	D
Test section				
Dimension (approx.)	55 m x 7.5 m	55 m x 8.5 m	55 m x 7.5 m	55 m x 7.5 m
Type of grid (product)	S&P Carbophalt® G			
Grid datasheet	OK (TDS)	OK (TDS)	OK (TDS)	OK (TDS)
Grid installation by company	Yes	Yes	Yes	Yes
Tack coat / Emulsion				
Emulsion type: proposal grid supplier	Polymer modified Emulsion (60%) Softening point: > 53 °C (Webacid Spécial C60BP4 C2)	unstable cationic emulsion of 70 % bitumen content (U70K)	Grisolast (Bitumen not emulsion)	HCB P4 → Nécessaire: Emulsion polymère
Dosage [g/m ² , residual binder]	180	approx. 1.0 kg/m ² depending on the evenness, if a 60% one will be used + 10 % amount (TOTAL, NOT RESIDUAL)	600	250
Comment	Maybe use of kalk. Depending of the temperature.	Important: Temps et conditions climatiques pour rupture de l'émulsion!	C recommande 600 g sur une surface fraîsée ce qui n'est pas réalisable avec une émulsion, ils posent eux-même un bitume Grisolast	Nous souhaitons pouvoir enchaîner la pose de la géogridle à la suite de l'application de l'émulsion
Final choice Emulsion	émulsion polymère 60% (type : Webacid Spécial C60BP4 C2 ou équivalent)		Grisolast	émulsion polymère 60% (type : Webacid Spécial C60BP4 C2 ou équivalent)
Dosage [g/m ²]	300	1100-1200	600 (voir ci-dessus)	420
Laying responsibility	Entreprise Kt. VS	Entreprise Kt. VS	C	Entreprise Kt. VS
Chipping / gravillonnage	No	No	Yes	Yes
Aggregate granulo.	-	-	vorumhüllt 8/11	6/8
Aggregate dosage [l/m ²]	-	-	6	approx. 4.0
Emulsion type	-	-	No Emulsion	HCB → Emulsion polymère
Emulsion dosage	-	-		1%
Laying responsibility	-	-	C	D
Comment			Le gravillonnage sera fait par SYTEC, la surface devra être roulée avant pose de l'enrobé (entreprise VS), pneu (pas de mixte)	Le gravillonnage sera fait par D
Grid laying plan				
Overlap length	Transvers. Min 20 cm Long. Min 10 cm	25 cm in length direction ; 15 cm in cross direction	longitudinal: env. 5 cm transversal: env. 15 cm	longitudinal: env. 5 cm transversal: env. 15 cm
Comment		-	Support sec et propre, température entre +5°C et 50 °C	Support sec et propre
Date information received	08.09.2021	04.09.2021	25.08.2021	18.08.2021 / 30.08.2021 / 22.09.2021

Table A5: Parameters of test track construction

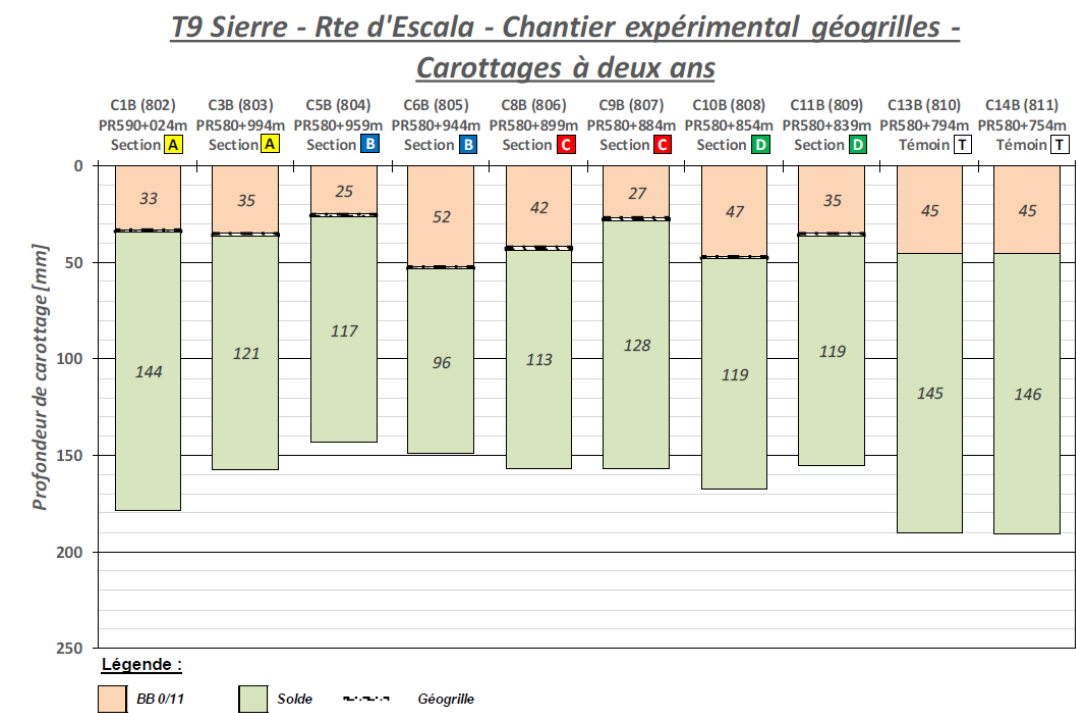


Figure A18: Extracted core layer thickness measurements

I ₂ road classification index		
Condition Index I ₂	Value sW [‰]	Assessment according to VSS 40 925 b
[0, 1]	< 1.8	Good
[1, 2]	[1.8, 3.0]	Medium
[2, 3]	[3.0, 4.6]	Sufficient
[3, 4]	[4.6, 6.5]	Critical
[4, 5]	≥ 6.5	Bad

Table A6: I₂ road condition index

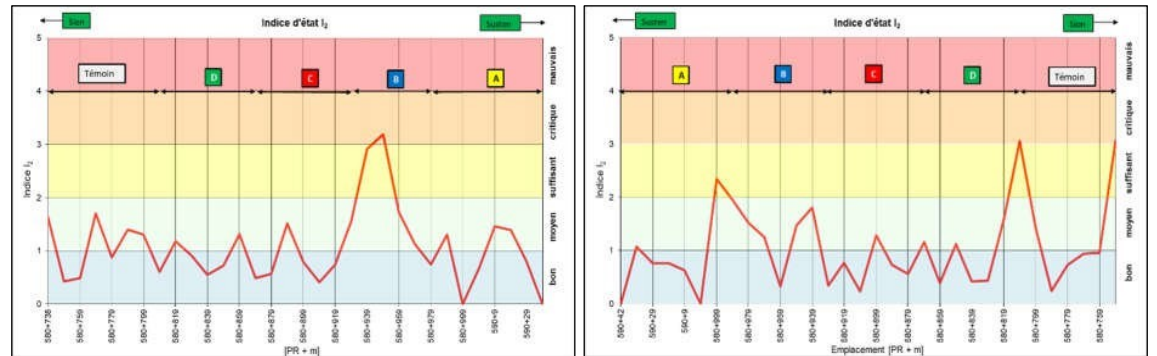
Note on special events: W-values at the level of particular events, such as manholes, keyholes, have been excluded from the analysis.

I ₃ road classification index		
Condition Index I ₃	Value sW [‰]	Assessment according to VSS 40 925 b
[0, 1]	< 5	Good
[1, 2]	[5, 8]	Medium
[2, 3]	[8, 12]	Sufficient
[3, 4]	[12, 18]	Critical
[4, 5]	≥ 18	Bad

Table A7: I₃ road condition index

Surface degradation Index I_1

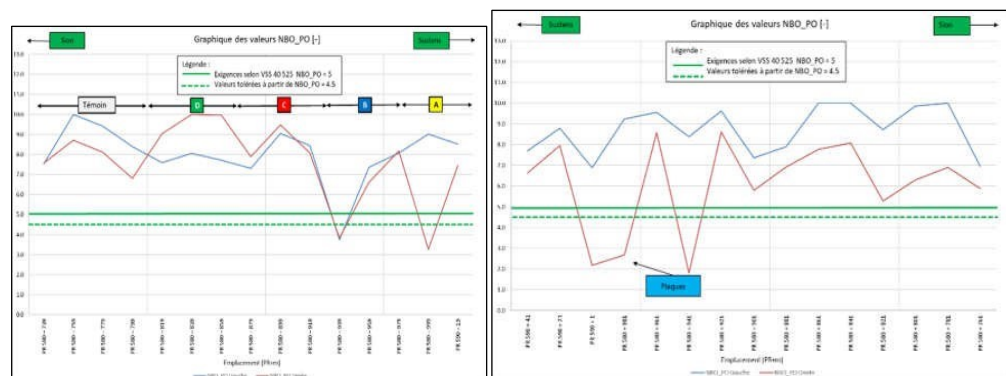
Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Average	0.9	1.1	1.4	1.2	1.4	1.1	0.9	1.3	1.3	1.1
Minimum	0.8	0.8	1.4	0.8	1.4	0.8	0.8	0.8	0.8	0.8
Maximum	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.4	0.8	1.4

Table A8: Details results of surface degradation Index I_1 **Figure A19:** Longitudinal flatness profile Index I_2 in direction Susten and Sion**Longitudinal Index I_2**

Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Average	1.1	1.0	1.0	0.5	0.8	0.9	2.2	1.4	1.0	0.7
Minimum	0.3	0.4	0.5	0.0	0.3	0.2	1.5	0.2	0.6	0.4
Maximum	1.6	1.4	1.5	1.0	1.8	2.1	3.2	2.2	1.5	1.0

Table A9: Details results of longitudinal Index I_2

The longitudinal smoothness of the roadway was also used with NBO values (National waveband). The results are as shown in Figure and Table A.

**Figure A20:** Longitudinal smoothness of road surface with NBO values in direction Sion and Susten

Longitudinal Index I2, NBO values									
Direction	NBO_PO left		NBO_PO right		NBO_MO left		NBO_MO right		
	<i>Susten</i>	<i>Sion</i>	<i>Susten</i>	<i>Sion</i>	<i>Susten</i>	<i>Sion</i>	<i>Susten</i>	<i>Sion</i>	
Minimum	4.0	7.3	4.7	6.8	8.9	-	9.3	-	
Average	7.9	8.8	7.9	7.7	8.9	-	9.3	-	
Maximum	10.0	10.0	10.0	8.6	8.9	-	9.3	-	
Standard deviation	1.4	1.1	1.3	0.7	-	-	-	-	
Requirements	4.5	4.5	4.5	4.5	5.5	-	5.5	-	
No. Of non-compliant values	1	0	1	0	0	-	0	-	
No. of values	14	7	14	7	1	-	1	-	
% of non-compliant values	7%	0%	7%	0%	0%	-	0%	-	

Table A10: Details results of longitudinal Index I2 NBO values

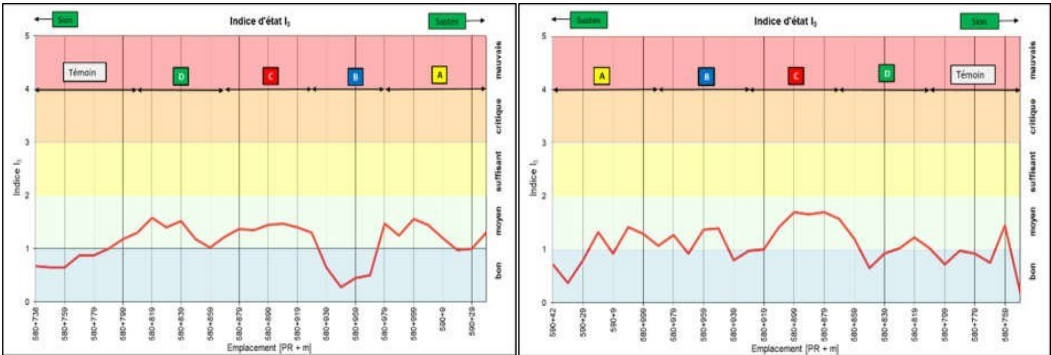


Figure A21: Trasverse flatness profile Index I₃ in direction Susten and Sion

Tranverse Index I ₃										
Section	Section RE		Section D		Section C		Section B		Section A	
Direction	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion	Susten	Sion
Average	0.8	1.0	1.4	1.2	1.3	1.2	0.6	1.2	1.3	0.9
Minimum	0.5	0.7	1.0	0.6	1.1	0.8	0.3	0.9	1.0	0.3
Maximum	1.1	1.4	1.6	1.5	1.5	1.7	1.3	1.5	1.5	1.5

Table A11: Details results of transverse Index I₃

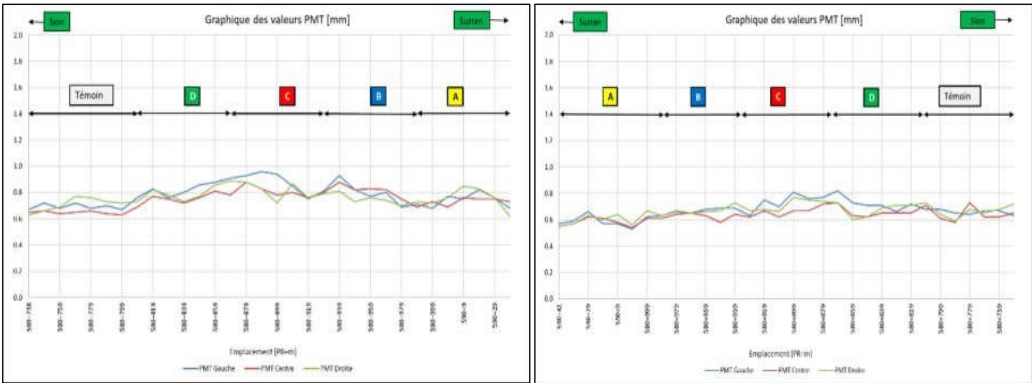


Figure A22: Profile of average texture depth in direction Susten and Sion

Values of average texture depth [mm]																														
Section	Section RE						Section D						Section C						Section B						Section A					
Direction	Susten			Sion			Susten			Sion			Susten			Sion			Susten			Sion			Susten			Sion		
Axis	G	C	D	G	C	D	G	C	D	G	C	D	G	C	D	G	C	D	G	C	D	G	C	D	G	C	D	G	C	D
Average	0.67	0.63	0.69	0.64	0.64	0.67	0.82	0.75	0.77	0.74	0.68	0.70	0.85	0.79	0.83	0.71	0.65	0.71	0.83	0.83	0.78	0.62	0.63	0.66	0.73	0.73	0.76	0.58	0.61	0.61
Minimum	0.61	0.61	0.64	0.59	0.59	0.60	0.78	0.69	0.74	0.68	0.64	0.66	0.76	0.72	0.75	0.63	0.62	0.61	0.79	0.81	0.76	0.58	0.59	0.63	0.67	0.66	0.68	0.48	0.57	0.52
Maximum	0.71	0.64	0.73	0.69	0.71	0.75	0.86	0.79	0.85	0.82	0.76	0.73	0.92	0.89	0.91	0.78	0.69	0.80	0.91	0.84	0.80	0.65	0.67	0.70	0.79	0.83	0.85	0.69	0.72	0.66

Table A12: Average texture depth value

11.5 Demonstration of grid installation

11.5.1 Summary of the Grid Installation Process of Product `A`

VSS 2019/422

«Einsatz von Asphaltbewehrungen zur Erhaltung von Strassen /
Utilisation de grilles de renforcement pour l'entretien des
chaussées »

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Document objective:

- This document aims at providing a brief overview of the grid installation demonstration of product `A` at BFH.
- The contained information will be further used for the preparation of laboratory samples in the laboratory.

Date: 21.07.2022

Version: 01

Author: Muhammad Rafiq Kakar, Nicolas Bueche

Pavement Structure

- ☐ **Pavement structures** for lab. tests → Proposed mix type:
 - ✓ AC 8 S (top layer) / AC B 11 S (bottom layer)
 - ✓ The layer thickness may vary from test to test.
- ☐ **Mixture production:**
 - ✓ Plant produced mixtures will be used
- ☐ **Materials:** aggregates, bitumen, filler → according to plant recipe
- ☐ **Recipe**
 - ✓ Plant/Producer

Type of Test and Specimens

1. Double Layer Slab Specimen

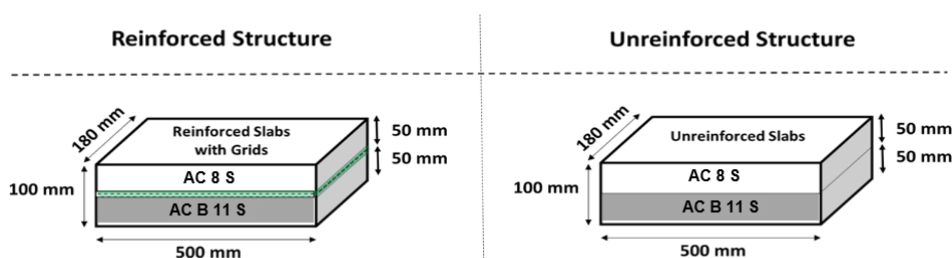


Figure A23: Size, shape and configuration of double-layer slab specimens with and without grid

2. Advance Shear Test (Task 2.5)

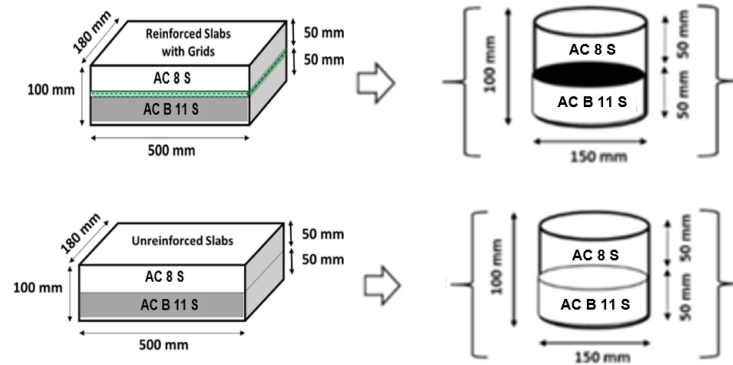


Figure A24: 150mm diameter cores drilled out of slab specimen

3. Crack Propagation Test (Task 2.2)

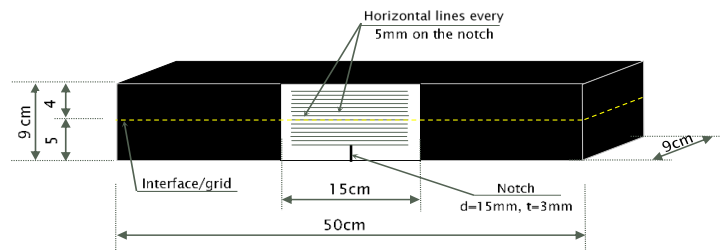


Figure A25: Size, shape and configuration of double-layer beam specimens with grid taken out from Slab

Date & Place of Demonstration

20.12.2021 (13:30 – 17:00 Uhr), BFH Burgdorf

Summary of the Grid Installation Process of Product `A`



Figure A26: Pictures taken during the demonstration process

Emulsion and grid product details

Location	Coating Material	*Dosage of Coating Material (Residual)	**Time for emulsion breakage	Grid Type	Chipping
Field Track	Bitex HCP Polymer modified Emulsion (60%)	180 g/m ²	Overnight	`A`	No
Laboratory	Bitex HCP Polymer modified Emulsion (60%)	180 g/m ²	3 hours	`A`	No

*Emulsion quantity of approximately ≈ 27 g with the help of syringe was applied on a slab surface of 0.09 m²

**Grid was installed after the complete breakage of the emulsion

Table A13: Details of emulsion and grid product

Important Note

The slab used for the demonstration purpose of grid installation as shown in the recording serves as dummy and will not be used in the real experiments. At BFH, the method of slab production is now modified to obtain a smooth surface at the interface specifically for the mixture AC B 11 (bottom slab).

`A` Grid Installation Process Details

Procedure and materials for the installation of the Grid `A` in laboratory conditions

Tack coat

The tack coat is an emulsion of PmB. The dosage of the emulsion should ensure 180g/m² of residual bitumen on the emulsified surface for the non-reinforced samples. The emulsion dosage with grids in the samples are given below. The tack coat is applied at a temperature of 50°C. Note that in the case of applying the emulsion on an asphalt concrete there is practically no contact of the emulsion with the mineral aggregate. So, in these conditions the emulsion rupture is governed by the evaporation of its aquatic phase.

The supplier of product `A` test its grids with a 60 % polymer modified tack coat (softening point 50 °C). One example of tack coat with the correct properties is given in below. The tack coat quality is important for the selected product `A` grids behavior.

Process for grid `A`

This process is for the following products:

Product 'A'

The sample production procedure follows the following steps:

- Fabrication and compaction of the bottom asphalt layer.
- 2h of curing in order for the asphalt concrete to cool down.
- Application of the tack coat (see details for tack coat below). Dosage: 180g/m² of residual bitumen (after evaporation of the water)
- 3h of curing. This curing time is just a preliminary consideration. If the rupture of the emulsion (complete water evaporation) occurs before of this proposed curing time and the surface is dry, then we can go to the next step.
- Installation of the grid. The grid should be installed only after the rupture of the emulsion (see instructions for grid installation below).
- Construction of the upper asphalt layer.
- 24 hours of curing.
- Cut/drill the samples.

Product 'A' Grids installation

After the compaction and the curing time of the bottom asphalt layer and after the application and the curing time of the tack coat, the installation of the grid is done according to the steps below:

- Cut the reinforcement grid. The dimensions of the cut grid will be equal to the plan dimensions of the asphalt concrete slab to be reinforced.
- Place the grid on the cracked tack coat.
- Lift up the grid and apply the gas blow torch from the bottom of the grid in order to burn the plastic film and melt the bitumen that the grid contains. When the plastic film has evaporated completely, the grid can be heated further until the bitumen has softened completely Recommendations:
 - o Do not use the full power of the blowing torch.
 - o Always keep minimum 10 cm distance between the torch and the material,
 - o Apply a constant-speed movement of the torch on each single strand of the grid; once in one direction and then in the other perpendicular direction. The speed of the movement should be such that allows a light melt of the bitumen contained in the grid.

The pictures here below give an idea about the grid installation procedure and the tools that can be used for this purpose.

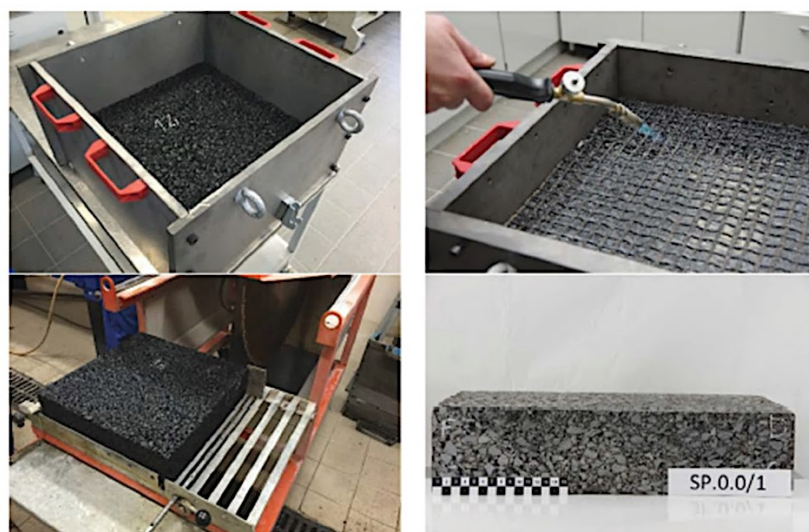


Figure A27: Grid installation process



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Technisches Merkblatt

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Erstellt am: 30.11.2015
Überarbeitet am: 22.01.2025
Gültig ab: 22.01.2025

Handelsname: **Webacid® C60B4**

TM-Nr.: F03011 - F08015

WEBACID® C60B4

Kationische, raschbrechende Bitumenemulsion für einen umweltfreundlichen Strassenbau

Begriff

Bitumenemulsionen sind umweltfreundlich. Es sind feinste Verteilungen von Bitumen, ein Naturprodukt in Wasser. Die für die Herstellung dieser Bitumenemulsionen verwendeten kationischen (sauen) Emulgatoren sind dank ihrer Aufbereitung und ihrer elektrochemischen Eigenschaften bestimmend für den spezifischen Charakter dieser Bitumenemulsionen.

Beschreibung

Webacid® C60B4 ist eine kationische, raschbrechende Bitumenemulsion gemäss Norm SN 670 205-NA EN 13808

- Webacid® C 60 B 4

Die Buchstaben-Zahlenfolge bedeutet:

- C	= kationisch (sauer)
- 60	= Bitumengehalt in %
- B	= Bitumen
- 4	= Brechwert ($\triangleq$ 110...195)

Webacid® C60B4 ist ein kalt und warm anwendbares Bindemittel. Es bindet auf trockenem Gestein schnell ab. Das Abbinden (Ende des Brechvorganges) ist dadurch gekennzeichnet, dass das Bitumen elektrochemisch am Gestein haftet, dass Wasser nach aussen verdrängt wird und anschliessend verdunstet. Ein späterer Wasserzutritt beeinträchtigt die Haftung des Bindemittels am Gestein nicht mehr.

Arbeiten mit **Webacid® C60B4** bringt folgende Vorteile:

- Mit **Webacid® C60B4** behandelte Strassen können, bei OB Anwendungen, sofort dem Verkehr freigegeben werden.
- Arbeitsunterbrüche und Verkehrsbehinderungen sind minimal.
- **Webacid® C60B4** hilft Kosten und Zeit zu sparen
- **Webacid® C60B4** ist ein umweltfreundliches und lösungsmittelfreies Produkt.

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 Technisches Merkblatt			
ctw	Erstellt am:	30.11.2015	Handelsname: <u>Webacid® C60B4</u>
	Überarbeitet am:	22.01.2025	
	Gültig ab:	22.01.2025	
			Seite 2 von 3
			TM-Nr.: F03011 - F08015
Anwendung			
Webacid® C60B4 kann gemäss den VSS-Normen für die Oberflächenbehandlungen, Tränkungen und Flickarbeiten verwendet werden. Die Durchführung einer erfolgreichen Arbeit mit Webacid® C60B4 verlangt eine vorherige Abklärung folgender Punkte: • Art und Zustand der Unterlage • Gefälle der Strassenfläche • Verkehrsbelastung • Lage im Gelände (Wald usw.) • klimatische Verhältnisse • Gesteinsart und Kornform des Splittes Nach diesen Gegebenheiten richtet sich die Webacid® C60B4 Dosierung. Auf einer bereits gebrochenen Emulsionsschicht ist die Benetzung des Abstreumaterials nicht mehr gewährleistet. Der Brech-, Koagulations- oder Abbindvorgang ist an der Bindemittelfarbe ersichtlich. Die anfängliche Farbe der Emulsion ist braun. Mit zunehmendem Durchbrechen wird die Oberfläche schwarz. Beim Verspritzen mit der Handlanze empfiehlt sich eine > 6 mm Düse. <u>Oberflächenbehandlung Typen E und D sowie Tränkungen Typ F</u> Oberflächenbehandlungen sind dünne, bituminöse Überzüge auf Strassenoberflächen. Detaillierte Angaben darüber sind in der SN 640 430 „Oberflächenbehandlungen“ enthalten. Werden einfache OB's durchgeführt, so ist es empfehlenswert, sich an die Bindemittel mit einem höheren Bindemittelgehalt zu halten, während bei doppelten OB's die niedrigeren den Vorzug haben sowie auch bei Tränkungen. Bei Temperaturen unter 5°C darf mit Bitumenemulsionen nicht mehr gearbeitet werden.			
Flickarbeiten			
Für die Ausführung von Flickarbeiten, welche dank Webacid® C60B4 unter dem Verkehr ausgeführt werden können, gelten die gleichen Richtlinien wie für Oberflächenbehandlungen und Tränkungen. Detaillierte Angaben über die Ausführung von Oberflächenbehandlungen sind in der SN 640 415d-NA EN 12271:2006 „Oberflächenbehandlungen“ enthalten.			
Technische Daten			
<u>Lieferform</u>		Fässer à 200 kg oder im Tankwagen	
<u>Lagerung</u>		2 Monate nach Herstellung resp. Verfalldatum auf dem Gebinde. <u>Vor Gebrauch das Fass kurz aufrühren, schütteln oder drehen.</u>	
<u>Dichte bei 25°C</u>		: ~ 0.99 g/cm³	
<u>Bindemittelgehalt</u>		: ca. 60%	
<u>pH-Wert</u>		: ca. 2.5	
<u>Verarbeitungstemperatur</u>		: > 5°C	
<u>Luft- und Bodentemperatur</u>		: > 5°C	
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			 1211



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Erstellt am: 30.11.2015
Überarbeitet am: 22.01.2025
Gültig ab: 22.01.2025

Handelsname: Webacid® C60B4

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TM-Nr.: F03011 - F08015

Technisches Merkblatt

Wichtige Hinweise

Kationische (saure) und anionische (alkalische) Bitumenemulsionen vertragen sich gegenseitig nicht. Daher ist es wichtig, dass Spritzgeräte und Behälter aller Art vor einem allfälligen Wechsel der Emulsionsart vollständig geleert und gereinigt werden.

Bitumenemulsionen sind frostempfindlich.
Bei Temperaturen unter 5°C sollte mit Bitumenemulsionen nicht mehr gearbeitet werden.

Bitumenemulsionen sind schwach wassergefährdend und sollten nicht in die Kanalisation oder Gewässer geleitet werden.

Der zum Einsatz gelangende Splitt muss hart sein, kubisch gebrochen und vor allem sauber und staubfrei sein (SN 670 103b-NA, Splitte für Oberflächenbehandlungen dürfen nicht mehr als 0,5 Massen-% Feinkorn kleiner 0,5 mm aufweisen).

Bei Anwendung spezieller Art und unter besonderen Bedingungen ist der Technische Dienst beizuziehen.

Die vorliegenden Angaben wurden aufgrund unseres derzeitigen Standes von Wissen und Erfahrungen erarbeitet. Wir garantieren die Lieferung von qualitativ einwandfreier Ware, können aber für eine unsachgerechte Anwendung und deren Ergebnisse keine Gewähr übernehmen.

Produktanwender müssen das jeweils neueste Technische Merkblatt unter www.ctwmuttertz.ch abrufen. Es gelten unsere allgemeinen Geschäftsbedingungen.
Für detaillierte Angaben konsultieren Sie bitte das aktuelle Sicherheitsdatenblatt, welches ebenfalls auf unserer Homepage unter www.ctwmuttertz.ch einsehbar ist.

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‘A’ Questionnaire for grid supplier – Laboratory tests

VSS 2019/422 «Einsatz von Asphaltbewehrungen zur Erhaltung von Strassen / Utilisation de grilles de renforcement pour l’entretien des chaussées »

Document objective: This document aims at collecting the necessary information/specification related to the (Grid) product provided by each grid supplier company.

Questionnaire for grid supplier – Laboratory tests

1. Product Supplier

Product ‘A’

2. Type of Grid for laboratory tests

2.1 Name and designation: Product ‘A’

2.2 Technical Data Sheet: Yes

Note: The proposed grid is intended to be placed in between top and binding layer (therefore, each supplier should recommend the type of grid that fits for this position). Additionally, the maximum top and bottom layer thicknesses (e.g. in case of slab specimen for MMLS3 test) is intended to be 4cm and 4cm, respectively.

3. Geometrical details of Grid

3.1 Mesh Dimensions: ~20 mm / ~20 mm

3.2 Thickness of threads: 3 – 5 mm

3.3 Based on type of Test (e.g. MMLS3, Crack propagation/overlap length), recommended grid orientation (if relevant, please make a scheme)

Note: The required amount of Grid for Lab. and Field test section from each supplier is roughly estimated to be approximately 650 square meters. However, this amount could be changed based on additional request.

4. Properties of Grid Materials

4.1 Composition: Glass / Carbon see table below from the product ‘A’

4.2 Tensile strength: see table below from the product ‘A’

4.3 Maximal deformation according to tensile strength: see Table A2 below from the product ‘A’ TDS

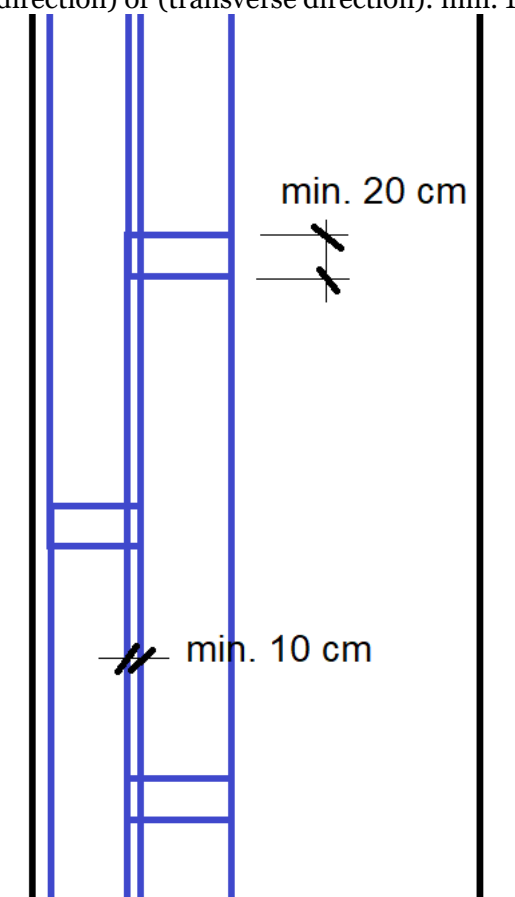
Product `A`		Longitudinal	Transverse
Mechanical properties	Unit	Glass fibre	Carbon fibre
E-Modulus	N/mm ²	≥ 73 000	≥ 265 000
Elongation	%	3 (±0.3)	1.5 (+0.2)
Tensile strength	kN/m	120	200
Fibre cross section	mm ² /m	46 (51 fibre strands)	46 (52 fibre strands)
CE-marking		According to EN 15381:2008	

Table A14: Grid product A properties

5. Proposed Grid Requirements

5.1 Proposed Grid Overlap Length (standard practice): min. 10 cm / min. 20cm

5.2 Proposed overlap direction i.e. along the traffic/loading (Longitudinal direction) or (transverse direction): min. 10 cm / min. 20cm



6. Important details of grid installation for each individual steps/instructions, for laboratory purpose

6.1 Steps for Installation of grid:

The sample production procedure follows the following steps (laboratory installation):

- Fabrication and compaction of the bottom asphalt layer.
- 2h of curing in order for the asphalt concrete to cool down.
- Application of the tack coat (see details for tack coat below) at 50°C. Dosage: 180g/m² of residual bitumen (after evaporation of the water)
- 3h of curing. This curing time is just a preliminary consideration. If the rupture of the emulsion (complete water evaporation) occurs before of this proposed curing time and the surface is dry, then we can go to the next step.
- Installation of the grid. The grid should be installed only after the rupture of the emulsion (see instructions for grid installation below).
- Application of Kalk (This only applies to field construction)
- Construction of the upper asphalt layer.
- 24 hours of curing.
- Cut/drill the samples.

“Grid installation”

After the compaction and the curing time of the bottom asphalt layer and after the application and the curing time of the tack coat, the installation of the grid is done according to the steps below:

- Cut the reinforcement grid. The dimensions of the cut grid will be equal to the plan dimensions of the asphalt concrete slab to be reinforced.
- Place the grid on the ruptured tack coat.
- Lift up the grid and apply the gas blow torch from the bottom of the grid in order to burn the plastic film and melt the bitumen that the grid contains. When the plastic film is completely burned the gas blow torch can be applied from the bottom of the grid in order to mild completely the grid bitumen. This point is delicate because the exposition of the binders to the flame action may result to excessive ageing of the material.

Recommendations:

- Do not use the full power of the blowing torch.
- Always keep minimum 10 cm distance between the torch and the material,
- Apply a constant-speed movement of the torch on each single strand of the grid; once in one direction and then in the other perpendicular direction. The speed of the movement should be such that allows a light melt of the bitumen contained in the grid.

The pictures here below give an idea about the grid installation procedure and the tools that can be used for this purpose.

6.2 Tag coat type, temperature and amount:

The tack coat is an emulsion of PmB 60 %. The dosage of the emulsion should ensure 180g/m² of residual bitumen on the emulsified surface for the non-reinforced samples. The tack coat is applied at a temperature of 50°C

6.3 Any other relevant detail regarding grid installation

7. Intended to participate for the installation of grid for the first production of samples in laboratory

7.1 Laboratory scale sample preparation

- Yes

11.5.2 Summary of the Grid Installation Process of Product `B`

Document objective:

- This document aims at providing a brief overview of the grid installation demonstration of product `B` at BFH.

Date & Place of Demonstration

27.01.2022 (08:30 – 12:00 Uhr), BFH Burgdorf



Figure A28: Pictures taken during the demonstration process

Emulsion and grid product details

Location	Coating Material	*Dosage of Coating Material (Residual)	**Time for emulsion breakage	Grid Type	Chipping	Description
Field Track	Webacid Special C60BP4 C2	570-660 g/m ²	3 hours	`B`	No	In comparison to test section installation, the emulsion dosage has been modified (depends on surface characteristics) while the other parameters have been kept constant
Laboratory	Bitex HCP Polymer modified Emulsion (60%)	480 g/m ²	3 hours	`B`	No	

*Emulsion quantity of approximately $\approx 72 (+2)$ g with the help of syringe was applied on a slab surface of 0.09 m^2 . A marginal safety factor of approximately ≈ 2 g is taken in to account for any loss of emulsion quantity during application making the amount of emulsion quantity as approximately ≈ 74 g in total. Furthermore it is recommended to weigh the brush before and after the installation to determine the loss in the brush.

**Grid was installed after the complete breakage of the emulsion

Table A15: Details of emulsion and grid product

DATA SHEET

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Product

Product description

Raw material

Type of coating

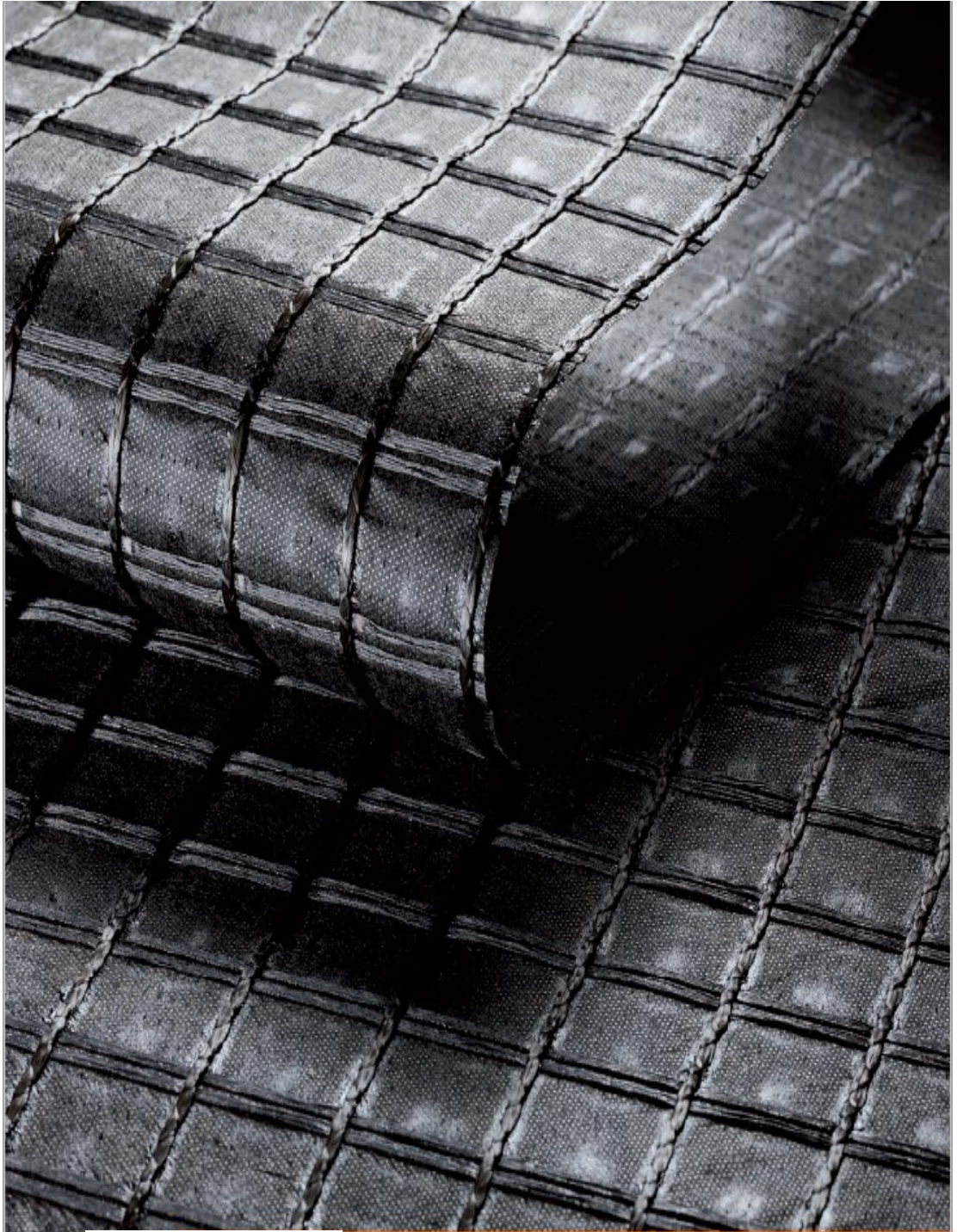
Colour

Environmental impact

Product `B`

Highly resistant asphalt reinforcing grid with
nonwoven and bituminous coating
recycled PET
polymer modified bitumen
black
non-hazardous

	unit	nominal value	tolerance
Mass per unit area DIN EN ISO 9864	g/m ²	270	-27
Bitumen content of the coating	%	65	-0
Tensile strength (MD) DIN EN ISO 10319	kN/m	50,0	-0,00
Tensile strength (CMD) DIN EN ISO 10319	kN/m	50,0	-0,00
Strain at nominal tensile strength (MD) DIN EN ISO 10319	%	12,0	+0,0
Strain at nominal tensile strength (CMD) DIN EN ISO 10319	%	12,0	+0,0
Tensile strength at 3 % strain (MD) DIN EN ISO 10319	kN/m	12,0	-0,0
Tensile strength at 3 % strain (CMD) DIN EN ISO 10319	kN/m	12,0	-0,0
Melting point reinforcing grid	°C	235	
Chemical resistance of the grid against solvents and de-icing salt		high	
Mesh size (MD)	mm	40,0	
Mesh size (CMD)	mm	40,0	
Available roll widths		100/ 200/ 300/ 400	
Standard length	m	150,00	
Standard width	cm	500,00	



Product `B`

Asphalt Reinforcement Grid

Installation Guidelines

1. Transportation and storage

Product 'B' asphalt reinforcement grid is supplied in roll form and packaged to protect against transport damage and site exposure. The rolls should be checked on arrival for any damage caused in transit and, if faulty, should be set aside.

The rolls should be stored on a clean and level base.

The wrapped material should be carefully moved around the construction site.

2. Surface preparation

Product 'B' must always be applied between two bituminous layers. A non-bituminous surface should be covered with a bituminous regulating layer. The high resistance of Product 'B' to mechanical damage allows an installation directly onto milled surfaces.



The surface should be prepared in accordance with recognised technical standards to ensure a good bond between the asphalt layers; it should be dry, clean and free of loose material.

The surface has to be even to ensure that the HaTelit® has full contact with the sublayer and that no voids are present.

A very uneven sublayer has to be either profiled or milled. Milled surfaces should not present channels deeper than 10 mm.

Loose residue should be removed.

Cracks of 3 mm width and less can be left untreated. However, cracks wider than 3 mm should be treated with a bituminous sealing after cleaning.

The prepared surface should be sprayed evenly with an unstable cationic emulsion of 70 % bitumen content (U70K) at a minimum rate of 0.6 kg/m² to ensure a good bond between the asphalt layers. On rough or open-textured surface the minimum rate needs to be further increased. On milled surfaces an amount of 1.0 – 1.2 kg/m² can be required.



When using a 60 % bitumen content the minimum rate of emulsion (considering the residual bitumen) needs to be adjusted accordingly.

The tack coat should be sprayed evenly.

When having warm weather conditions Product 'B' should be placed onto the broken emulsion. When having cool weather conditions, Product 'B' should be placed into partly-broken emulsion.

The emulsion break can be recognised by a colour change from brown to black. Ensure all spraying conditions of the tack coat are followed.

3. 'B' installation

The asphalt reinforcement grid should be pulled out over the prepared surface and stay in a flat, crease-free condition.



The unrolling can be undertaken with the help of a simple system, either manually, or mechanically when using wider, heavier rolls. The grid should be pulled equally on both sides of the roll, in order to prevent the development of folds and creases. To ensure a good adhesion, 'B' should be pressed onto the base during or after the installation, e.g. using a street broom. If necessary 'B' can be additionally pressed down using a lightweight tandem roller.



An installation plan should be considered for difficult and complicated area shapes.

To accommodate gullies and channels 'B' can be cut with a knife. The grid-covered surface should not carry normal road traffic until the covering asphalt course has been placed.



In the length direction the jointing of the rolls should be with a 250 mm overlap. The end of a roll should always be placed over the beginning of the next, ensuring the overlap is not lifted during the asphalt covering operation.



In the cross direction the overlap should be 150 mm. When placing several roll widths side by side the joints should be staggered. The overlaps should be treated with a minimum of 0.15 kg/m² of bitumen emulsion.

4. Asphalt laying

The normal specifications and procedure are followed for placing the asphalt course.

'B' has to be covered with a minimum 40 mm thick asphalt layer (in compacted condition).

However, when using reinforcing grids special attention should be paid to the quality of the asphalt and the installation in order to achieve a high-quality asphalt layer. Laying the asphalt should follow the installation of the 'B' without delay.

Before placing the subsequent asphalt course, the tack coat should be completely broken, with all solvent and water having evaporated. Both the underlying base layer and the reinforcement must be dry. If it rains after installation special steps must be undertaken.



The reinforcement grid must be laid free of folds and waves. However, if a slight wave builds up in front of the paving machine, this will not detrimentally affect the performance of the reinforcement. Joints in the asphalt pavement must not coincide with overlaps in the rolls of reinforcement grid. The current technical requirements for the compaction of asphalt layers must be observed when using 'B'.



The paver and delivery vehicles should move carefully over the grid-covered surface to avoid grid displacement. Sharp turns, rapid changes in speed and hard braking should be avoided, when moving to the front of the paver. The supply vehicle in front of the paver should be driven and not pushed.

5. Conclusion

The installation of a reinforced asphalt layer should follow normal technical procedures. A few additional points have to be borne in mind when using 'B' and the installation crew should be informed. We are freely available to answer further questions, particularly covering special applications. We reserve the right to introduce changes and improvements to the product and installation guidelines to ensure the advance of technical progress. No warranty claims can be derived from these instructions.

A summary of the most important points covering the installation of Product `B`

1. Surface preparation

Ensure the surface is clean, dry and free of any loose material.

2. Tack-coat spraying

An unstable cationic emulsion of 70 % bitumen content (U70K) should be evenly applied at a rate of $0.6 \text{ kg/m}^2 - 1.2 \text{ kg/m}^2$ (to be adjusted on site); adequate enough to hold down the `B`.

3. Tack-coat 'break'

`B` shall be placed on the partly broken – or, in warm weather conditions, on the broken – emulsion (colour change from brown to black).

4. Laying the grid

The grid should be pulled out over the surface by using a pulling system. The `B` roll has to lay on the sublayer, while being unrolled, and not hang in the air. To ensure a good adhesion, `B` should be pressed onto the base during or after the installation, e.g. using a street broom. Laying of the asphalt (minimum 40 mm in compacted condition) should follow the installation of the `B` without delay. If it rains after installation special treatment must be undertaken.

5. Joints/overlaps

In the cross direction 150 mm and in the length direction 250 mm. The end of a roll always being placed over the beginning of the next, ensuring the overlap is not lifted during the covering operation. The overlaps should be treated with a minimum of 0.15 kg/m^2 of bituminous emulsion.

6. Bends and curves

On curves `B` is cut to shorter lengths and laid with overlaps. The length of the pieces depends on the radius of the curve.

7. Trafficking the grid

The grid-covered surface should not carry normal road traffic until the covering asphalt course has been placed. When moving vehicles on the HaTelit®, sharp turns, rapid changes in speed and hard braking must be avoided.

Note:

Special measures are required in case of very high ($> 30^\circ\text{C}$) or very low ($< 5^\circ\text{C}$) air temperatures and for steep gradients.

Product `B` Questionnaire for grid supplier – Laboratory tests

Document objective: This document aims at collecting the necessary information/specification related to the (Grid) product provided by the grid supplier company.

Questionnaire for grid supplier – Laboratory tests

1. Name of Supplier

1.1 Product `B`

2. Type of Grid for laboratory tests

Note: The proposed grid is intended to be placed in between top and binding layer (therefore, each supplier should recommend the type of grid that fits for this position). Additionally, the maximum top and bottom layer thicknesses (e.g. in case of slab specimen for MMLS3 test) is intended to be 4cm and 4cm, respectively.

- **according to our installation guideline the thickness must have 4 cm or more**

3. Geometrical details of Grid

3.1 Mesh Dimensions: 40/40 mm

3.2 Thickness of threads: ca. 1,9 mm

3.3 Based on type of Test (e.g. MMLS3, Crack propagation/overlap length), recommended grid orientation (if relevant, please make a scheme)

Note: The required amount of Grid for Lab. and Field test section from each supplier is roughly estimated to be approximately 650 square meters. However, this amount could be changed based on additional request.

- **to deliver the best possible width we need the exact road width**

4. Properties of Grid Materials

4.1 Composition: recycelt PET

4.2 Tensile strength: 50/50 kN/m

4.3 Maximal deformation according to tensile strength: < 12%

5. Proposed Grid Requirements

5.1 Proposed Grid Overlap Length (standard practice):

5.2 Proposed overlap direction i.e. along the traffic/loading (Longitudinal direction) or (transverse direction): s. installation guideline

25 cm in length direction – roll on roll

15 cm in cross direction – roll widths on roll widths

6. Important details of grid installation for each individual steps/instructions, for laboratory purpose

6.1 Steps for Installation of grid: s. attached installation guideline

6.2 Tag coat type, temperature and amount: s. installation guideline

The prepared surface should be sprayed evenly with an unstable cationic emulsion of 70 % bitumen content (U70K) at a minimum rate of 0.6 kg/m² to ensure a good bond between the asphalt layers. On rough or open-textured surface the minimum rate needs to be further increased. On milled surfaces an amount of 1.0 – 1.2 kg/m² can be required.

Depending on the situation in the laboratory test (evenness etc.) the amount could be even less.

If only a cationic emulsion of 60 % bitumen content is available the amount had to be increased by 10%.

6.3 Any other relevant detail regarding grid installation: s. installation guideline

7. Intended to participate for the installation of grid for the first production of samples in laboratory

7.1 Laboratory scale sample preparation

- Yes

11.5.3 Summary of the Grid Installation Process of Product `C`

Document objective:

- This document aims at providing a brief overview of the grid installation demonstration of product `C` at BFH.
- The contained information will be further used for the preparation of laboratory samples in the laboratory.

Date: 14.09.2022

Date & Place of Demonstration of Product `C`

23.06.2022 (14:00 – 17:00 Uhr), BFH Burgdorf

Summary of the Grid Installation Process



Figure A29: Picture taken during the demonstration process

Emulsion and grid product details

Location	Coating Material	*Dosage of Coating Material (Residual)	**Time for emulsion breakage	Grid Type	Chipping	Description
Field Track	Smart Tape (pre-bituminised 1000 g/m ²)	-	-	*Product `C` (Patch, pre-bituminised 1000 g/m ²)	No	In comparison to test section installation, the same product was used; only the pre-bituminised dosage has been adapted according to discussion between BFH and Product `C` supplier.
Laboratory	Smart Tape (pre-bituminised 350 g/m ²)	-	-	**Product `C` (Patch, pre-bituminised 350 g/m ²)	No	

**As on the construction site, the patch also needs the normatively required installation temperature of the pavement in the laboratory.*

***Product `C` with highly modified bitumen membrane was initially planned but has not been used for logistical reasons.
Asphalt temperature min. 140 °C*

Table A16: Details of emulsion and grid product

Important Note

The slab used for the demonstration purpose of grid installation as shown in the recording serves as dummy and will not be used in the real experiments. At BFH, the method of slab production is now modified to obtain a smooth surface at the interface specifically for the mixture AC B 11 (bottom slab).

Product `C` Grid Installation Process Details

Selbstklebende, lokale Asphaltarmierung

Einfach, sicher und schnell – die dauerhafte Sanierung von lokalen Belagsrissen wird mit der innovativen Asphaltarmierung von `C` leicht gemacht. Product `C` Patch besteht aus einem Glasfasergitter, das bestehende Risse überbrückt, und einer stark selbstklebenden

Bitumenmembrane (SAMI), die als Haftvermittler wirkt. Auf diese Weise werden vier Funktionen gleichzeitig gewährleistet: selbstklebend für die einfache Installation, guter Schichtverbund, Abdichtung im Belag und Verhinderung von Belagsrissen (Reflexionsrissen).

Vorteile

- Stark selbstklebend, ohne Erhitzen
- Schnelles und sicheres Verlegen
- Haftvermittler und Membrane (SAMI) inklusive
- Hoch wirksam gegen Rissbildung

Product `C` Patch eignet sich für die Sanierung von lokalen Rissen und kleineren Belagsflächen. Das Produkt ist auf glatten sowie gefrästen Belagsoberflächen bestens einsetzbar. Für grössere Flächen empfiehlt sich die Anwendung der Asphaltarmierungssysteme Product `C`.



Einbau

1. Vorbereitung

- Belagsfläche nass reinigen
- Schlaglöcher und Ausbrüche mit bituminösem Mischgut füllen
- Maximale Fräsrillentiefe 6mm
- Grössere Risse nach Bedarf mit Bitumen vergiessen



Verlegen SYTEC Gridseal® Patch

2. Armierung

- Product `C` Patch direkt auf der staubfreien und trockenen Fläche positionieren und ausrollen
- Schutzfolie abziehen
- Anwalzen mit Pneuradwalze



Anwalzen

3. Belag

- Asphaltbelag einbauen (mindestens 4 cm Stärke)
- Fertig!



Einbau Belag



Sanierung



Sanierung



Sanierung Autobahn N2 Riviera-Lugano



Sanierung Strassenrand
(Nahtstelle alte/ neue Tragschicht)



Sanierung Quartierstrasse Buchs AG



Rissanierung Schächte

Product `C` Questionnaire for grid supplier – Laboratory tests

Document objective: This document aims at collecting the necessary information/specification related to the (Grid) product provided by grid supplier company.

Questionnaire for grid `C` supplier – Laboratory tests

1. Name of Supplier

1.1 Product `C`

2. Type of Grid for laboratory tests

Note: The proposed grid is intended to be placed in between top and binding layer (therefore, each supplier should recommend the type of grid that fits for this position). Additionally, the maximum top and bottom layer thicknesses (e.g. in case of slab specimen for MMLS3 test) is intended to be 4cm and 4cm, respectively.

3. Geometrical details of Grid

3.1 Mesh Dimensions:

approx. 25 x 25 mm

3.2 Thickness of threads

approx. 1 mm

3.3 Based on type of Test (e.g. MMLS3, Crack propagation/overlap length), recommended grid orientation (if relevant, please make a scheme)

The product `C` patch is intended for local-surface installation and acts biaxially - in both directions equally. The grids panels are overlapped by 5 cm lengthwise and 10 cm crosswise. If individual cracks are to be repaired, we recommend an anchoring length of min. 50 cm on both sides.

Note: The required amount of Grid for Lab. and Field test section from each supplier is roughly estimated to be approximately 650 square meters. However, this amount could be changed based on additional request.

4. Properties of Grid Materials

4.1 Composition:

Glass fiber strands (E-glass) coated with polymer-modified coating and a highly self-adhesive membrane.

4.2 Tensile strength:

min. 100 kN/m on both sides (CMD/MD)

4.3 Maximal deformation according to tensile strength:

max. 3%

5. Proposed Grid Requirements

- 5.1 Proposed Grid Overlap Length (standard practice):
5 cm lengthwise and 10 cm crosswise
- 5.2 Proposed overlap direction i.e. along the traffic/loading (Longitudinal direction) or (transverse direction):
Roof shingle like along paving direction

6. Important details of grid installation for each individual steps/instructions, for laboratory purpose

- 6.1 Steps for Installation of grid:
On smooth surfaces, dry and clean, surface temperature between +15° C and + 50° C.
1. Remove the protective film
 2. Roll out without wrinkles
 3. Roll on with a pneumatic wheel roller
 4. Lay the asphalt surface.
- 6.2 Tack coat type, temperature and amount:
The tack coat is already integrated in product `C` Patch. It is not necessary to pre-spray the surface with tack coat.
- 6.3 Any other relevant detail regarding grid installation
Asphalt temperature min. 140 °C, thickness min. 4 cm thickness.

7. Intended to participate for the installation of grid for the first production of samples in laboratory

- 7.1 Laboratory scale sample preparation

Yes

Questionnaire for grid supplier – Laboratory tests

Document objective:

- This document aims at providing a brief overview of the grid installation demonstration of Texum product at BFH.
- The contained information will be further used for the preparation of laboratory samples in the laboratory.

Date: 24.10.2022

Version: 01

Author: Muhammad Rafiq Kakar, Nicolas Bueche

Date & Place of Demonstration of Grid Product `D`

20.12.2021 (08:30 – 12:00 Uhr), BFH Burgdorf

11.5.4 Summary of the Grid Installation Process of Product `D`



Figure A30: Pictures taken during the demonstration process

Emulsion and grid product details

Location	Coating Material	*Dosage of Coating Material (Residual)	**Time for emulsion breakage	Grid Type	Chipping	Description
Field Track	Webacid Special C60BP4 C2	420 g/m ²	3 hours	Product `D`	Yes	In comparison to test section installation, the emulsion dosage has been modified (depends on surface characteristics) as well as the choice to work without chipping for the laboratory tests.
Laboratory	Bitex HCP Polymer modified Emulsion (60%)	250 g/m ² *	3 hours	Product `D`	No	

*Emulsion quantity of approximately ≈ 37.5 g with the help of syringe was applied on a slab surface of 0.09 m². A marginal safety factor of approximately ≈ 2 g is taken in to account for any loss of emulsion quantity during application making the amount of emulsion quantity as approximately ≈ 39.5 g in total.

**Grid was installed after the complete breakage of the emulsion

Table A17: Details of emulsion and grid product

Important Note

The slab used for the demonstration purpose of grid installation as shown in the recording serves as dummy and will not be used in the real experiments. At BFH, the method of slab production is now modified to obtain a smooth surface at the interface specifically for the mixture AC B 11 (bottom slab).

Grid Product `D` Installation Process Details

Méthode d'application de la géogridde

1. Surface d'application

- Si nécessaire, procéder à un reprofilage de la chaussée à l'aide d'un enrobé bitumineux.
- Colmater les fissures de plus de 4 mm avec un produit adapté, tel qu'une masse de remplissage de joints.
- Nettoyer soigneusement la surface de la chaussée pour éliminer toute saleté ou débris pouvant nuire à l'adhérence de la géogridde.

2. Enduit d'accrochage

- Le choix du type d'enduit et de son dosage doit prendre en compte les conditions climatiques, l'état du support et la rugosité de la surface.
- Appliquer l'enduit de manière uniforme pour garantir une adhésion optimale de la géogridde.

3. Géogridde

`D`

- Appliquer la géogridde en respectant les chevauchements préconisés.

3. Pose de la couche d'enrobé

Caractéristiques de la géogridde `D`

Composition	Verre	Carbone	
Résistance à la traction	> 120	> 200	<u>kN/m</u>
Allongement à la rupture*	3.5 - 4.5	1,6	%
Module d'élasticité*	> 80'000	> 240'000	<u>N/mm²</u>
Dimensions des mailles	18 X 18		<u>mm</u>
Géotextile	Polyester, 18 g/m2		
Masse surfacique	229		<u>g/m²</u>
Largeur	1.00		<u>m</u>
Longueur	50		<u>m</u>
Poids	11		<u>kg</u>
Couche d'accrochage	250 à 400 g/m2 de liant résiduel		

Product `D` Questionnaire for grid supplier – Laboratory tests

Document objective: This document aims at collecting the necessary information/specification related to the (Grid) product provided by grid supplier company.

Questionnaire for grid `D` supplier – Laboratory tests

1. Name of Supplier

1.1 Product `D`

2. Type of Grid for laboratory tests

Note: The proposed grid is intended to be placed in between top and binding layer (therefore, each supplier should recommend the type of grid that fits for this position). Additionally, the maximum top and bottom layer thicknesses (e.g. in case of slab specimen for MMLS3 test) is intended to be 4cm and 4cm, respectively.

3. Geometrical details of Grid

3.1 Mesh Dimensions:
approx. 18 x 18 mm

3.2 Based on type of Test (e.g. MMLS3, Crack propagation/overlap length), recommended grid orientation (if relevant, please make a scheme)
The product `C` patch is intended for local-surface installation and acts biaxially - in both directions equally. The grids panels are overlapped by 5 cm lengthwise and 10 cm crosswise. If individual cracks are to be repaired, we recommend an anchoring length of min. 50 cm on both sides.

Note: The required amount of Grid for Lab. and Field test section from each supplier is roughly estimated to be approximately 650 square meters. However, this amount could be changed based on additional request.

4. Properties of Grid Materials

4.1 Composition:
E-CR glass, Carbon and polyester sail

4.2 Tensile strength:
E-CR Glass > 120 kN/m and Carbon > 200 kN/m

4.3 Maximal deformation according to tensile strength:
E-CR = 3.5 – 4.5% and Carbon 1.6%

5. Proposed Grid Requirements

- 5.1 Proposed Grid Overlap Length (standard practice):
5 cm lengthwise and 10 - 15 cm crosswise
- 5.2 Proposed overlap direction i.e. along the traffic/loading (Longitudinal direction) or (transverse direction): see point 5.1

6. Important details of grid installation for each individual steps/instructions, for laboratory purpose

- 6.1 Steps for Installation of grid:
See annex IV for details.
- 6.2 Tack coat type, temperature and amount:
HCB P4 250 gram residual, installation after breakage
- 6.3 Any other relevant detail regarding grid installation: --

7. Intended to participate for the installation of grid for the first production of samples in laboratory

- 7.1 Laboratory scale sample preparation
 - Yes or No: Yes

Acknowledgements

The authors would like to thank the Canton of Valais, for providing the test section and conducting the field measurements and evaluation of the extracted cores. The authors also would like to acknowledge the contributions for the part of the image analysis of the 4PB beam crack evaluation carried out by Dr. Mohammad Nishat Akhtar and Assoc. Prof. Dr. Elmi Abu Bakar, Department of Aerospace Engineering, Universiti Sains Malaysia.

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Project conclusion



Schweizerische Eidgenossenschaft
Confédération suisse
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Confederaziun svizra

Eidgenössisches Departement für
Umwelt, Verkehr, Energie und Kommunikation UVEK
Bundesamt für Strassen ASTRA

FORSCHUNG IM STRASSENWESEN DES UVEK

Version vom 09.10.2013

Formular Nr. 3: Projektabschluss

erstellt / geändert am: 22.01.2025

Grunddaten

Projekt-Nr.: VSS 2019/422

Projekttitel: Einsatz von Asphaltbewehrungen zur Erhaltung von Strassen

Enddatum: 22.01.2025

Texte

Zusammenfassung der Projektergebnisse:

In this project, several laboratory and field tests (construction of test tracks, unreinforced RE, reinforced section A, B, C and D) were carried out in collaboration between BFH and Empa to compare the performance of different reinforcement systems to the unreinforced situation. The interlayer bond test revealed that there are reinforcement grids, which in the laboratory lead to better or comparable behavior compared with unreinforced specimens, while in situ, due to construction constraints and difficulties, the reinforcement grids always seem to provide a barrier between layers leading to lower interlayer bond properties. Nevertheless, there are reinforcement systems that can achieve the required average value of 15 kN according to the Swiss standard. The AST results equally showed that a reinforcement grid has a negative influence on the interlayer bonding. The effect of moisture damage showed that the strength decreases due to water conditioning of the specimens.

The crack propagation tests conducted with the MMLS3 traffic load simulator showed that system A outperforms all others, while system D performs poorly, carrying only about 60% of the loads compared to system A. According to deformation sensors (Lvd1) measurements system A provided high stiffness to the structure, whereas system D seems even weaker than the reference without reinforcement. This fact is extremely interesting since both systems consist of the same reinforcement material type including material stiffness and geometry.

The 4PB beam crack propagation test conducted at 5 °C revealed that all reinforced specimens sustained a higher number cyclic loading irrespective of the grid type in comparison to the unreinforced reference specimens. The specimens with grid reinforcement B offered the highest resistance against crack propagation followed by system D. The specimens with a chosen overlap length of 15 cm were found to be more resistant in the 4PB test, compared to the full-length specimens. This shows that overlapping of the tested reinforcement grids according to the suppliers' recommendations does not provide a problem and that the over-lap length for a specific reinforcement product could even be optimized. The modelling developed in the frame of this research project will allow for determining the performance of different reinforcement systems using product data provided by the reinforcement suppliers. Using this tool the amount of testing could be reduced significantly, and different test conditions could easily be evaluated.

Finally, the life cycle cost analysis results show that the use of asphalt reinforcement can provide an additional benefit regarding a long-term analysis period of 50 years. If the asphalt reinforcement is not installed optimally, an additional maintenance measure may be necessary, leading to increased costs within the life cycle. However, the use of asphalt reinforcement can have an advantage in the application case if the technically sensible replacement of all asphalt layers is temporarily not feasible for traffic, economic, personnel or other reasons.



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Eidgenössisches Departement für
Umwelt, Verkehr, Energie und Kommunikation UVEK
Bundesamt für Strassen ASTRA

Zielerreichung:

The main goal of the project set to investigate the performance of selected reinforcements grids compared to the conventional method without reinforcement was accomplished within the given budget.

Folgerungen und Empfehlungen:

All evaluations and tests performed during the project revealed that the number of laboratory and field specimens must be increased to be able to provide reliable findings and conclusions. This is specially true regarding standardization processes and LCCA methods. The road owners and contractors must ensure that the necessary requirements for pavement construction are strictly met when installing reinforcement systems and that all conditions for an optimal installation of specific reinforcement grids are fulfilled. Additionally, a close collaboration between contractors and grid suppliers is essential. When looking at the performed crack evaluation in the laboratory, the MMLS3 crack propagation test must be considered more realistic and comparable to the field situation compared to the 4PB test (though its a small-scale test and easy to be perform) due to its load configuration and geometry of the specimens, especially regarding the mesh size of the reinforcement products. Therefore, it is recommended to conduct this type of medium-scaled test, whenever possible using deflection measurements with Lvot's and digital image correlation (DIC) data analysis to understand the mechanism of reinforcement systems, determining horizontal and vertical crack propagation patterns. To reduce the amount of testing and evaluate different test conditions, the modelling developed in the frame of this research project should be applied for determining the performance of different reinforcement systems. In this respect it is essential that the reinforcement suppliers provide all necessary input data. In accordance with previous works, this research project demonstrated the ability of reinforcement grids to prevent reflective cracking in general, and it is therefore recommended to use such reinforcement systems for rehabilitation and maintenance measures. Since it was shown that there is a strong connection between the interlayer bonding properties and their ability to prevent reflective cracking on the one hand interlayer bond testing should be performed for benchmarking and ranking different systems, on the other hand suppliers of reinforcement grids need to optimize and/or improve their products in this respect. It would be interesting to conduct a survey, at least visual inspection and crack intensity measurements even after completion of the project to have more comprehensive analysis and outcome of the final conclusions. For field comparison of different reinforcement products, a test track with equal test sections in terms of traffic, alignment etc. would be desirable.

Publikationen:

None

Der Projektleiter/die Projektleiterin:

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Unterschrift des Projektleiters/der Projektleiterin:



Schweizerische Eidgenossenschaft
Confédération suisse
Confederazione Svizzera
Confederaziun svizra

Eidgenössisches Departement für
Umwelt, Verkehr, Energie und Kommunikation UVEK
Bundesamt für Strassen ASTRA

FORSCHUNG IM STRASSENWESEN DES UVEK

Formular Nr. 3: Projektabschluss

Beurteilung der Begleitkommission:

Beurteilung:

Eines der Ziele lag in der Evaluation der auf dem Markt erhältlichen Produkte. Dies wurde sowohl in Testfelder wie auch im Labor in grosser Bearbeitungsbreite durchgeführt. Mittels FEM-Simulationen (Finite Elemente) konnten die im Labor gemessenen Werte für die verschiedenen Geotextilien nachvollzogen werden. Ein weiteres Ziel des Projektes war es, Grundlagen für eine Überarbeitung der SN-Norm zur Dimensionierung von Strassenaufbauten zu erarbeiten. Es zeigte sich, dass die Anzahl Testfelder und Laborproben zu klein waren um dieses Ziel zu erreichen. Der Bericht zeigt auf, wie dies in einer weiteren Forschungsarbeit erreicht werden könnte.

Umsetzung:

Eine Umsetzung in die Praxis bzw in die Normierung kann nicht erfolgen, da die Ergebnisse für den gesamten Markt zu wenig repräsentativ sind.

weitergehender Forschungsbedarf:

Da das Ziel der Erarbeitung von Grundlagen zur Dimensionierung von Strassenaufbauten unter Verwendung von Geotextilien nicht erreicht wurde, bleibt diesbezüglich ein weiterer Forschungsbedarf bestehen.

Einfluss auf Normenwerk:

keines

Der Präsident/die Präsidentin der Begleitkommission:

Name: Angst

Vorname: Christian

Amt, Firma, Institut: IMP Bautest AG, Oberbuchsitzen

Unterschrift des Präsidenten/der Präsidentin der Begleitkommission: