THE RELATIONSHIP BETWEEN ROAD SURFACE PROPERTIES AND ENVIRONMENTAL ASPECTS

Master of Science Thesis

By

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FINAL REPORT

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This thesis has been carried out in cooperation with KOAC-NPC.

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PREFACE

This study has been performed as a partial fulfilment of the requirements for the degree of Master of Science in Road and Railway Engineering at Delft University of Technology. The activities of this research have been performed at KOAC-NPC (graduate internship). KOAC-NPC is a large independent company with research facilities and expertise in a wide range of mobility infrastructure.

The study presented in this research describes the relationship between road surface properties of Dutch higher order road networks and environmental aspects. Traffic safety, fuel consumption and traffic noise are the three main aspects related to the environment which are investigated in this study.

I would like to use this opportunity to thank the following persons for their assistance, contribution and support during this research. First of all I would like to thank Christ van Gurp for the opportunity I got to perform this research at KOAC-NPC and also for his daily guidance. Furthermore I would like to thank my graduation committee (T. Scarpas, M. Haas, L. Houben and Christ van Gurp) for their time, guidance and support during this research. Special thanks go to Lambert Houben for his kind contribution in this research and with general course issues which I encountered during my study at Delft University of Technology. Furthermore I would like to thank Jacob Groenendijk for his extended contribution by sending me quite a lot of relevant documents related to this study and for providing me with quite a lot of information and practical knowledge in road engineering. I also want to thank Paul de Valk, Freek Hol, Arco Blanken of KOAC-NPC for providing me with the required data. Also thanks go to the whole staff of KOAC-NPC in Apeldoorn and Nieuwegein for their assistance and support during this research. This work would not have been possible without the guidance and assistance of all the previously mentioned persons.

Finally I would like to thank my family in Aruba for their support and encouragement during this research. I greatly appreciate the time and effort given by them and all the above mentioned persons during this research. Without their support it would have not been possible to accomplish this research.

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SUMMARY

Sustainable development is nowadays an essential concept which affects all sectors of our society. Undoubtedly, the road construction and the mobility sector are also challenged by this concept. Due to this fact, the government and the road authorities should strive for sustainable (environment-friendly) roads and sustainable mobility. In order to achieve this goal, a sustainable policy must be conducted to find a balance between the three principles People, Planet and Prosperity. Within the context of sustainable development and sustainable mobility, road construction should reduce the negative effects that it has on people and environment. This requires a safer, greener and an economical transportation system.

By reviewing the previously mentioned aspects safer, greener and economical, it can be seen that the road surface (properties) has a major impact on these aspects. First of all, the traffic safety is strongly affected by the road surface condition. Secondly, the fuel consumption and the generated traffic noise are both influenced by the road surface. This is due to the interaction between the road surface and the vehicles.

This study has been performed in order to develop a model that can be used by road engineers as a tool for getting an indication of what they can expect from designed asphalt pavements in terms of crash rate, fuel consumption, traffic noise and pavement surface characteristics. By developing such a model, it can be considered that the first step towards designing environment-friendly roads has been done. Before developing the model, knowledge on the background of the road surface properties, traffic safety, fuel consumption and traffic noise is required. This is required in order understand the relationships between the road surface properties and the environmental aspects (traffic safety, fuel consumption and traffic noise). Furthermore, knowledge on the background of the methods used to measure the road surface properties are also required. Information regarding these topics can be found in this study. In order to be able to determine the crash rate due to a water film layer on the road surface a method has been invented in this study. In this method, the skid resistance has been used as the indicative parameter. The principles behind this method can be found in this study.

The necessary data for this study is obtained for a major part from the KOAC-NPC database and the Strategic Highway Research Programme of The Netherlands (SHRP-NL) database. These databases consist of motorway and provincial road sections. Besides these two databases, data of three other road sections A1 (Apeldoorn), N732 (Lonneker-Losser) and A70 (Bamberg, Germany) are used. These data are used to determine the performance (polishing rate) of different mineral aggregates used in the most applied wearing courses in the Netherlands (SMA, DAC & PAC).
By processing the data from the previously mentioned databases, several performance models are obtained indicating how the road surface properties (skid resistance, texture, rutting, roughness and crossfall) develop with time or traffic. Each performance model is described with a function which is used in the end to develop the final model. The relationships between the road surface properties and the environmental aspects are either described with a function or a matrix.

This study reveals that the proportion of the total crash rate in which the skid resistance is responsible for, is much higher than for the four other road surface properties in the analysis. This indicates that the expected crash rate of designed asphalt pavements is mainly determined by the road surface property skid resistance. Furthermore, from the evaluated mineral aggregates in this study, the mineral greywacke yields the best performance (friction life). By means of this it can be concluded that greywacke is the most durable mineral aggregate. However it must be said that the decrease of the friction coefficient due to the passing axle loads does not correlate with the polished stone value of the mineral aggregates.
The results from this study also reveal that the mean profile depth (MPD) for dense asphalt wearing courses (DAC & SMA) appears to increase with time or traffic whereas it decreases for porous asphalt wearing courses.

Furthermore, in the Netherlands, pavements are so well designed and maintained that the development of roughness is slow in comparison to third world countries (lower requirements and much higher intervention values). Most of the roads in the Netherlands would not reach an IRI-value of 3.5 m/km within a period of 20 years. This means that changes in roughness will certainly have negligible effect on the crash rate. In other countries where much higher IRI values are encountered, it is expected that the roughness will have a much bigger impact on the crash rate.

Regarding the findings of this study about the fuel consumption and the traffic noise, it can be concluded that the fuel consumption increase with 1.1% due to an increase of the texture depth with 0.4 mm. Due to the fact that PAC wearing courses associate with greater texture depth, it can be concluded that the fuel consumption is the highest on PAC wearing courses. This is the least on DAC wearing courses.
The increase of the traffic noise with time or traffic on a DAC wearing course is higher than that of a SMA wearing course. This conclusion is based on the development of the MPD. According to an alternative modelling approach described in this study, it has been found that PAC wearing courses have an annual increase of the traffic noise with 0.18 dB.

The final conclusion of this research programme is that this study resulted in a tentative model that can be used by road engineers as a tool to design environment-friendly roads and to get an indication of what can be expected from designed asphalt pavements in terms of crash rate, fuel consumption and traffic noise.

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LIST OF SYMBOLS AND ABBREVIATIONS

AADT	Annual Average Daily Traffic
AEC	Asphalt Emulsion Concrete
ARAN	Automatic Road Analyser
CBA	Cost Benefit Analysis
CPB	Controlled Pass-By
CR	Crash Rate
CROW	Centrum voor Regelgeving en Onderzoek in de Grond-, Water- en Wegenbouw
DAC	Dense Asphalt Concrete
dB(A)	Decibel (A-weighted)
DID	Data – ICT – Dienst
DLPA	Double Layer Porous Asphalt
EAPA	European Asphalt Pavement Association
ETD	Estimated Texture Depth
FWD	Falling Weight Deflection
HSRP	High Speed Road Profiler
IPG	Innovatie Programma Geluid
IRI	International Roughness Index
LBC	Locked Braking force Coefficient
LCA	Life Cycle Analysis
LTPP	Long Term Pavement Performance program
MDO	Material Damage Only
MPD	Mean Profile Depth
MTD	Mean Texture Depth
PAC	Porous Asphalt Concrete
PSV	Polished Stone Value
RAW	Stichting Rationalisatie en Automatisering in de Grond-, Water- en Wegenbouw
RD	Rut Depth
RMS	Root Mean Square
RRC	Rolling Resistance Coefficient
RWS	Rijkswaterstaat
SCRIM	Sideway Force Coefficient Routine Investigation Machine
SFC	Sliding Friction Coefficient
SHRP	Strategic Highway Research Program
SMA	Stone Mastic Asphalt
SMTD	Sensor Mean (Measured) Texture Depth
SPM	Statistical Pass-by Method
SPTD	Sand Patch Texture Depth (2/3 * SMTD)
SR	Skid Resistance
SRM	Standard Calculation Method (Standard Reken Methode)
SWOV	Foundation for Scientific Research into Traffic Safety (Stichting Wetenschappelijk Onderzoek Verkeersveiligheid)
TD	Texture Depth
TRL	Transport Research Laboratory

1. INTRODUCTION

1.1 Background

Sustainable development is nowadays a very important concept in our society which has to be taken into consideration. Since the introduction of this concept, many obligations have been created for the government and for local companies. Not knowing this concept has in general consequences for everyone. This is also the case for road construction and the mobility sector which are being challenged by sustainable development. This is the reason why the government and the road authorities nowadays strive for sustainable roads (environment-friendly) and sustainable mobility. The only way to accomplish this goal is by conducting a sustainable policy in order to find a balance between the three principles of sustainable development. These three principles of sustainable development are the following: People, Planet and Prosperity. The principle 'People' refers to the social acceptance, the principle 'Planet' to the environmental considerations and finally the principle 'Prosperity' to the economic progress and growth. This PPP's concept is a practical and widely used tool to give the concept of sustainable development hands and feet. This concept should be applied in order to achieve sustainable roads and mobility. When these three principles of sustainable development overlap each other and at the same time are in balance, one can speak of sustainable development. The following figure 1-1 depicts the three principles of sustainable development.

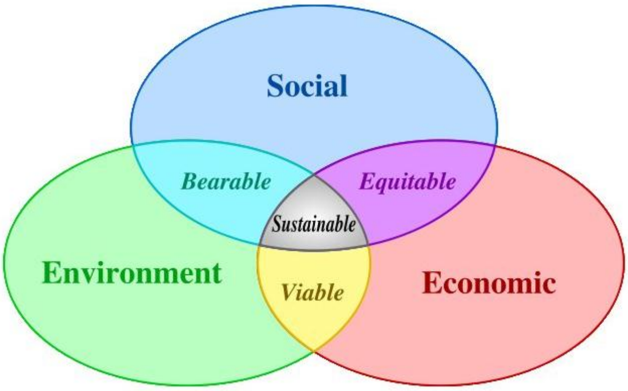


Figure 1-1: The three principles of sustainable development: social, environment and economic (Source: Wikipedia).

The road network is for many years worldwide known as the most intensively used transport system. This vital network must deal with the continuously growing demand for transport of people and goods. According to many other studies done before, the road network will be the leading type of transportation system also for the coming years. This will also be the case in the Netherlands. In order to achieve sustainable roads and sustainable mobility it is for the road authorities (Rijkswaterstaat and Provinces) of great importance to pay more attention to all the different parts of a pavement structure. Within the context of sustainable development and sustainable mobility, road construction should become more effective. At the same time it should reduce the negative effects that it has on people and environment. This requires a safer, greener and an economical transportation system (see figure 1-2). In the following paragraphs these aspects will be elaborated further to make clear what these three mentioned aspects imply.

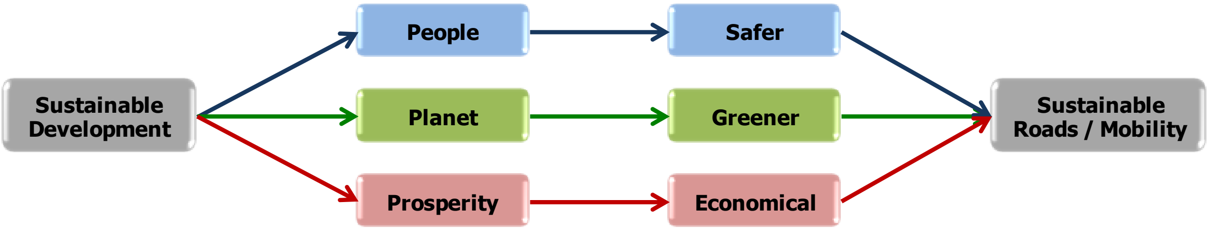


Figure 1-2: Sustainable roads/mobility in relation with the three principles of sustainable development.

Safer: A safer transportation system refers to an infrastructure where as a matter of priority work has to be done to the infrastructure in order to prevent road accidents. This will reduce the related social costs. In addition to this, a safer transportation system should also mitigate the severity of road accidents. Before striving for safer roads, knowledge on the background of road accidents should firstly be obtained. This will be discussed later in this report.

Greener: With a greener transportation system is referred to a system with less air pollution and less traffic noise. The noise caused by the contact between the tires and the road surface is a serious problem especially in urban areas. Traffic noise can in some cases be harmful to people's health. Another important aspect that belongs to a greener transportation system is the energy consumption. With the energy consumption is not referred to the energy that is required for the construction, maintenance or operation of the road but to the energy that is consumed by the traffic throughout the pavement life. The fuel consumption by the traffic is partly determined by the road surface.

Economical: Designing economical roads implies that roads have to be built by spending as less as possible. Most of the time men do not realize how important the quality is that will be provided by the roads during their service life. Therefore it is always important when designing a new road to first weigh the offered quality against the costs that provide that particular quality (cost benefit analysis). By doing this it can be seen if it is really worthwhile to construct that specific road. Another aspect which effects the costs as well as the quality of designed pavements is the type of material that is used. The reason for this is because each type of material will offer its own particular quality. The influence on the cost can be explained as follow: a material that has to be imported from abroad is much more expensive than one of equal quality that can be retrieved locally. Besides that, a material that does not need to be imported is much greener because it leads to less CO_2 emissions due to the reduced transportation of the material (shorter transportation distance).

Taking a look back at the last three mentioned aspects, one can see that the road surface has a major impact on all these three aspects. Road surfaces in bad condition can in some cases be the main cause of traffic accidents. Road surfaces have an influence on the fuel consumption and on the traffic noise that is produced during the contact between the tires and the road surface. Furthermore, the durability of road surfaces is a very important aspect related to the 'economical' factor because durable road surfaces lead to less maintenance. Less maintenance means also less cost. As mentioned before all the three aspects SAFER, GREENER and ECONOMICAL can be correlated to the terms durability and sustainability. Both durability and sustainability of pavement structures are greatly influenced by the quality at which pavement structures perform during their life. For example: a low quality level leads to a lower service life that also brings negative consequences for the use of materials, traffic safety, fuel consumption and nuisance for the environment and the road users.

This research aims to relate the developments of the road surface properties with the sustainability/environmental aspects traffic safety, fuel consumption, traffic noise and durability (figure 1-3). This will be performed in order to develop a model or tool that can be used to evaluate the total performance of road surfaces. In other words, this model can be used by road engineers to design Safer, Greener and Economical pavements in the future. In the next paragraphs an extensively elaborated overview will be given on what will be exactly researched.

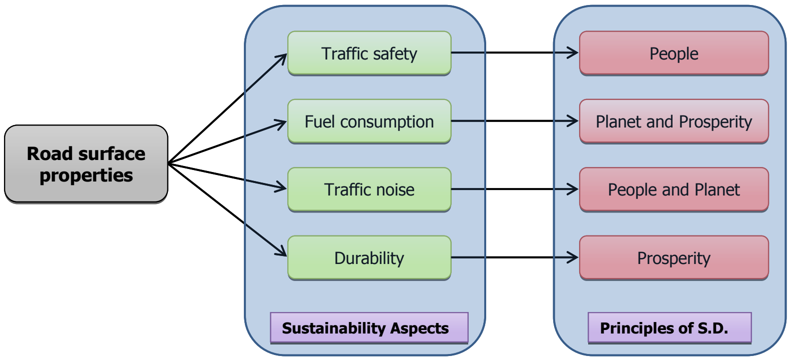


Figure 1-3: The relationship between road surface properties and sustainability/environmental aspects.

1.2 Research Description

In this part of the report the project will be formulated. In the design or redesign process of asphalt pavements it is very important for road engineers to know in advance what can be expected from a designed asphalt pavement. Issues that road engineers want to know in advance from designed road surfaces are the following: the expected service life, the expected crash rate, the expected fuel consumption by the traffic and the expected traffic noise during the pavement life. Crash rate is the number of traffic accidents for a reference number of vehicle kilometres. By knowing in advance how long, approximately, the wearing course will perform above a certain acceptance quality level, it will also be possible to know in advance how many times maintenance is required within a certain period. The expected crash rate, fuel consumption and traffic noise are factors that are greatly influenced by the quality of the road surface. A low quality level leads to a shorter service life and early maintenance is required. This also leads to negative consequences for the use of materials, traffic safety, energy consumption and nuisance for the environment and the road users. So it is very important for the road authorities to ensure that the roads are always in good condition because they are the ones who are responsible for the quality of the roads they manage. This make it clearer why quality care is a very important aspect to road surfaces. Quality care has two advantages. First of all, it makes the risks manageable. Secondly, it guarantees optimum durability and sustainability.

The main purpose of this study is to develop a predictive model in which several data of new asphalt pavements can be entered in order to calculate how safe, environment-friendly and durable they will be during a certain period of time. Road engineers can use this as a tool for designing new roads. After all this model will enable road engineers to calculate in advance what they can expect from new designed asphalt pavements. For already constructed pavements this model can also be used to evaluate the total performance of these pavements with regard to the crash rate, fuel consumption, traffic noise and service life.

Another problem that can be solved by the results of the study is the following. A road engineer has for instance to choose between three types of wearing course where each type has its own quality decline during a period of 20 years (see figure 1-4). Each type of wearing course contains different materials. Type A and B have a high initial quality level but for type A maintenance is required after 10 years. The quality of type B wearing course also decreases with time but more gradually with no intermediate maintenance required. Type C has a lower initial quality level in comparison with the other two types but decreases with time linearly very slowly. After 20 years the material still complies with the minimum acceptance level. After 20 years, it also has a higher quality level than the other two. Now the big question is: which one of these three types of wearing course performs the best within a period of 20 years.

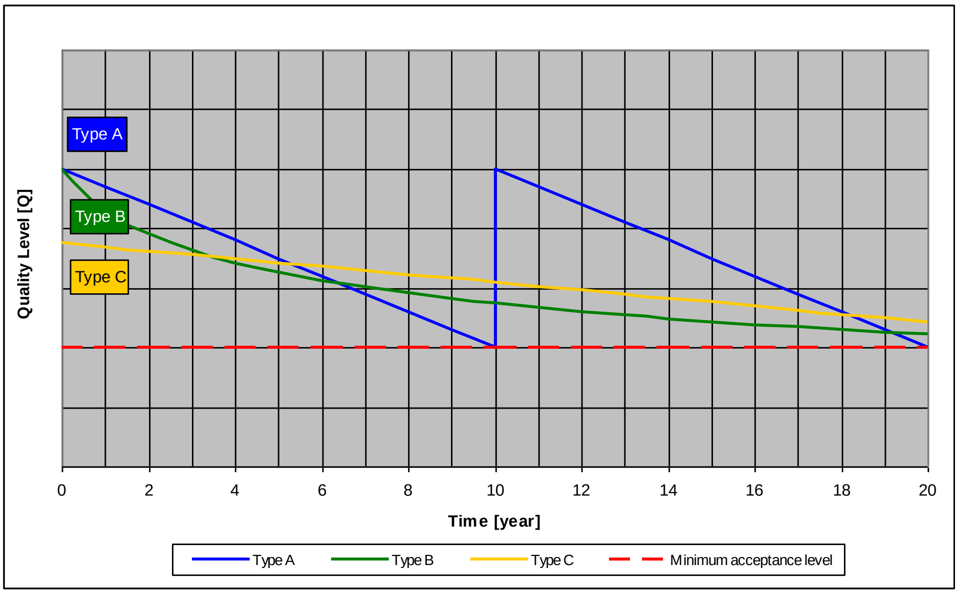


Figure 1-4: Problem that can be solved by the results of this study.

1.3 Research Objectives and Research Challenges

1.3.1 Research Objectives

The main research objective is formulated as follow:

- ***Develop a model that can be used by road engineers as a tool for getting an indication of what they can expect from designed asphalt pavements in terms of crash rate, fuel consumption, traffic noise and pavement surface characteristics.***

The research objectives are formulated as follow:

- What is the relationship between the road surface properties and the following aspects: crash rate, fuel consumption and traffic noise.
- Obtain more knowledge on the background of the aspects traffic safety, fuel consumption and traffic noise.
- How do road surface characteristics develop with time or traffic.
- Determine the intervention levels for all relevant road surface characteristics for main roads and provincial roads.
- Determine the pavement surface friction life of different types of mineral aggregates.
- How often is maintenance required if road engineers decide to use mineral aggregates with less resistance to polishing.
- What effect has the type of mineral aggregate in the wearing course on the durability of the road surface (number of years performing above the acceptance level).
- Which road surface property is the most important with regard to the traffic safety.
- Which road surface properties have impact on the fuel consumption and traffic noise.
- Determine what savings may be achieved for the Dutch road network and find out where the biggest savings can be realized (in terms of lowering the number of fatal accidents and amount of consumed fuel).
- Make clear how important traffic safety, fuel consumption, traffic noise and maintenance are for achieving sustainable (environment-friendly) roads and mobility and also make clear how important the monitoring of road surface characteristics is.

1.3.2 Research Challenges

The biggest challenge in this research is to find relationships between the road surface characteristics and the crash rate that can be used or applied for the Dutch situation. The crash rate is defined as the number of traffic accidents per one million kilometres. Some of these relationships can be found in studies performed earlier but others are not readily available for the Dutch situation. In these cases relationships found in other countries comparable to the Dutch situation might provide a solution. Because Dutch roads are much more safe in comparison with many other countries, care must be taken in simply copying foreign relationships.

Another challenge in this research is to find out to what extent the road itself is the main cause of traffic accidents. The road is not the only component of the road transport system that can be involved in traffic accidents. The driver and the vehicle are the other two components that may also be responsible for the cause of road accidents. So in order to get more reliable results at the end of this research, it will not be ideal to assume that the component 'road' is responsible for all the road accidents. More data should be sought for values in other studies that give a more realistic indication for what share the road is responsible for the total number of accidents.

1.4 Research Approach and Research Scope

1.4.1 Research Approach

The research programme consists of three phases.
In the first phase, also named the initial phase, a literature study will be performed and hypotheses will be drafted. In this initial phase appropriate databases will be selected too.
In the second phase the collected information will be studied and analysed and the data will be processed. The processing of the data will result in a number of mini-models. At the end, they will be combined in forming the final model. The last step to be carried out in the second phase is the analysis of the mini models.
In the last phase the processed and analysed data will be discussed. Furthermore, the sensitivity of the final model to certain parameters or assumptions will be investigated. When this part is completed, the final model can be verified to check whether it predicts proper results. The sensitivity analysis will be performed by changing only one parameter at the time to see what effect this change has on the result.
At the end of this study, relevant conclusions will be drawn and recommendations given. The research approach of this research programme with the three contained phases is depicted in figure 1-5.

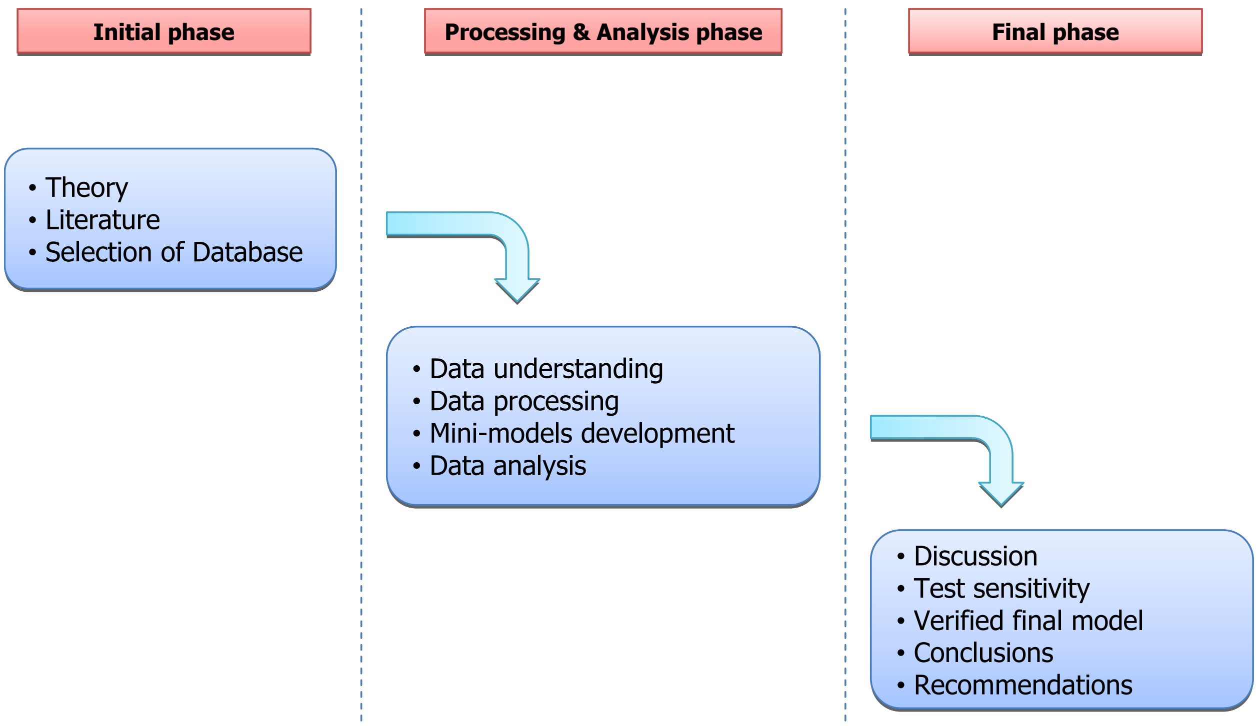


Figure 1-5: Research Approach with the three phases (initial, processing & analyzing and final).

1.4.2 Research Scope

The research scope encompasses development of a practical and consistent model that can be used by road engineers for designing durable and environment-friendly but also safe asphalt pavements. The main focus will be laid on trying to design safer, more economical and environment-friendly asphalt pavements with sufficient long service lives with limited maintenance. The model should also take into consideration which mineral aggregates and other materials may be used in the wearing course. The entire process for realizing the final model will be described and evaluated in this study.

1.5 Report Structure

This report consists of several chapters following the research approach discussed in section 1.4.1 and depicted schematically in figure 1-5.

Chapter two will present the survey on the research objectives. This chapter gives an overview of the research goal by describing what the final developed model should be able to do. The data input and the expected results from this model will be described as well.
In chapter three an overview of the theory and literature used in this study will be given on the road surface characteristics which will be evaluated and categorised. This chapter will also provide in what kind of parameters the road surface properties must be expressed and how they are to be measured. Furthermore, the review of the theory will be presented to see what the critical road surface properties are and what effect they have on the environmental aspects traffic safety, fuel consumption and traffic noise. Previous studies will also be used for obtaining relationships between road surface properties and the environmental aspects mentioned before.
Chapter four presents what data will be used and how they will be processed for obtaining a series of mini-models that will be put together to develop a final model being the main objective of this study.
In chapter five the research method will be discussed. This chapter describes the general limitations, assumptions and considerations supporting the research method.
In chapter six the results of the performed data analysis will be presented (case analysis).
Chapter seven presents the developed model and described how this developed model works.
Finally, in chapter eight conclusions of this research will be given as well as recommendations for further studies. Furthermore, a discussion and reflection on the results of this study will also be given in this chapter.

In addition, figure 1-6 presents a general scheme of the report structure with the two main issues of this study. These two issues are the following:
A: Relationship pavement property vs. sustainability (environmental) parameter.
B: Change of pavement property with time or traffic.

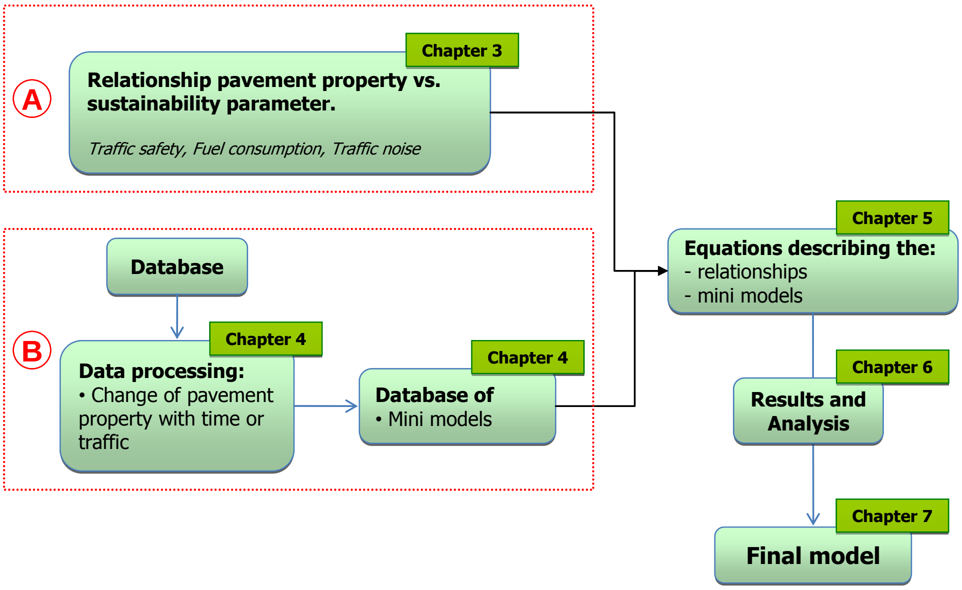


Figure 1-6: General scheme of the report structure.

2. SURVEY OF THE RESEARCH OBJECTIVE

The objective of the whole study is to develop a model with which durable and environment-friendly road surfaces can be designed. This chapter will highlight the survey performed on the research objective by describing several components of the model that needs to be developed. These are:

- how should the model be developed
- what should the model be capable of
- what data should be entered into the model
- how should the model process the inputted data
- what results may be expected as outcome of the model so that at the end it can be concluded that the targeted research goals have been achieved.

2.1 How should the model be developed

The first step in development of the model consists of acquiring knowledge on the background of road surface properties, traffic accidents, fuel consumption, traffic noise and durability of road pavement surfaces. After acquiring this knowledge, the next step contains gathering as much data as possible of measurements performed on asphalt pavements with regard to the road surface properties. The properties of a road surface and the measurement methods used in the Netherlands for measuring these properties are described in chapter 3. By processing the data several performance models will be obtained indicating how the road surface properties will develop with time or traffic. These performance models will be called mini-models in this study.

By relating the deterioration of the road surface properties (mini models) with others properties found in the literature such as crash rate, pavements may be assessed in terms of risk on expected crash rates. This can be carried out in the same way for the others environmental aspects i.e. fuel consumption and traffic noise. By combining all the mini-models a larger single model can be developed for evaluating the total performance of road surfaces for sustainable development. This model can be used in the design phase of asphalt pavements.

2.2 What should the model be capable of

As mentioned previously, the purpose of the model to be developed is to help road engineers in designing safer, greener and durable road surfaces. The 'safety level' of a designed road surface will be expressed in terms of crash rate that may be expected for a particular road and road surface. Fuel consumed by traffic expected to travel on a designed road surface will be a measure for indicating how 'green' the designed road and road surface are. Traffic noise might also be such a measure. By choosing which mineral aggregate will be used for the wearing course of the designed pavement, an indication can be given of how long this material will comply with the acceptance level requirements and how 'green' the end solution will be. The 'green level' of the mineral aggregate is predominantly governed by the source of the material and the handling of the material.

2.3 What data should be entered in the model

The data that should be entered in the model is divided in two parts: data on the designed pavement and data expressing the traffic and climate. The data input are listed in table 2-1.

Table 2-1: List of data that should be entered in the model.

Pavement data:	**Traffic data:**
• Skid resistance	• Annual Average Daily Traffic (AADT)
• Type of wearing course	• Growth rate
• Type of used mineral aggregate	• Percentage truck traffic
• Roughness	• Number of traffic lanes (one direction)
• Thickness layer	• Speed limit
• Stiffness modulus/type of subgrade	• Design life
• Rut depth	
• Crossfall	**Weather condition data:**
• Texture depth	• Rain intensity

2.4 How should the model process the inputted data

The model will be developed in Microsoft Excel. The input data will be fed into an Excel spreadsheet from which the processing and presenting of the main results will be performed automatically on the basis of the earlier entered mini-models (performance models) of the road and road surface properties. All the performance models will be described as functions of a number of variables that can be entered in the spreadsheet.

2.5 What results may be expected as outcome of the model

The expected crash rate, fuel consumption, traffic noise and service life (mainly pavement friction life) are the four principal outcomes from the model(s). In other words, the total performance of the designed road and road surface will be performed in their entirety on the three aspects of sustainable development (social, environment and economic). Durability of the asphalt pavements will be expressed in the number of years that the pavement will perform above the acceptance level. The sustainability of asphalt pavements (how environment-friendly) will be expressed in terms of crash rate, fuel consumption and traffic noise. At the end this model can be used as a tool by road engineers to design SAFER, GREENER and ECONOMICAL roads (environment-friendly and durable roads).

3. THEORY AND LITERATURE OVERVIEW

This chapter presents a summary of the theory and literature overview of road surface properties and the environmental aspects with respect to these road surface properties. The road surface properties that will be evaluated in this study are categorised and extensively described in this chapter. The methods used for measuring the road surface properties are also discussed. Furthermore, this chapter will indicate with which parameters the condition of the road surface properties are expressed and with which requirements Dutch roads have to comply with. The most influential factors and components leading to changes to the road surface properties will also be discussed in this chapter. The last section of this chapter provides for the link between the road surface properties and the environmental aspects.

3.1 Road Surface Properties

This section specifies what may be considered as road surface characteristics. Road surface as well as the whole pavement structure must meet many requirements. Both road users and road authorities demand quite a lot from a road pavement. The main requirements for the road users are traffic safety, traffic noise, fuel consumption and driving comfort. On the other hand, for the road authorities the aspects durability and traffic safety are the main two requirements. Road authorities demand from road pavements that they have to be durable (long lasting pavements). This means that they have to perform for a long period above a certain level of quality (minimum level). Road surface properties are determined by the composition of the materials used in the pavement structure and the pavement design. The pavement design is referred to the following properties: layer thickness, mineral aggregate characteristics such as particle size, gradation and shape, and the degree of compaction. Due to the repeated interaction between the traffic and the road surface, changes may and will take place to the properties of the road surface. The result of these changes have most of the time negative consequences for the traffic safety, fuel consumption, durability of the road surface or for the other previously mentioned requirements for the road users.

Road surface properties can be divided in three categories.
First of all, the category skid resistance being the most important one because traffic safety is the main criterion. Sufficient friction between the vehicle tires and the road surface is vital, because otherwise vehicles will not be able to brake within a short distance in case of emergency.
Secondly, the longitudinal surface properties of a road pavement form the second category. To this category belong the two following properties: the texture and the roughness of a road surface. Because the skid resistance is determined by the level of angularity and asperity (texture) of the road surface, the texture is primarily linked to the skid resistance. This will be further explained in sub-section 3.1.1 skid resistance.
The transverse properties of a road pavement form the last category. This category consists of wheel path rutting and the transverse slope (crossfall) of a road pavement. The previously mentioned five road surface properties which will be discussed in this study are shown in figure 3-1. The following sub-sections will describe the five properties of the road surface in more detail.

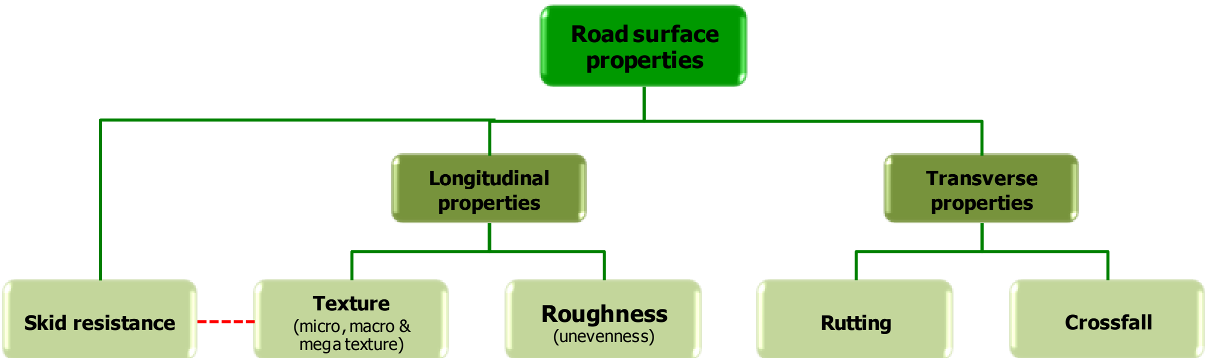


Figure 3-1: Scheme of the five road surface properties that will be discussed in this research.

3.1.1 Skid Resistance

The skid resistance of a road surface is one of the most important requirements that road authorities must consider when building, maintaining and managing road pavements. This surface characteristic has a very strong link with traffic safety. Skid resistance is for a major part governed by the properties of the mineral aggregate at the pavement surface. This not only applies to a wearing course of an asphalt mix but also to a chip sealed surface. The skid resistance can simply be described as the resistance to sliding or in other words, the degree of friction between a tire and the pavement surface. For that reason, skid resistance is expressed as a friction coefficient. The level of skid resistance is affected by many variables. These variables can be grouped in four groups. These are the aggregate factors, environmental factors, load factors and vehicle factors. These factors and their matching group are presented in table 3-1.

Table 3-1: The factors of the four groups which affect the level of skid resistance.

Group	Factors
(1) Aggregate	• Aggregate surface (micro and macro texture) • Geological properties of aggregate • Aggregate size and shape • Type of wearing course
(2) Environmental	• Water film thickness • Surface contamination (especially in tunnel) • Temperature • Rainfall effects
(3) Load	• Age of pavement surface • Traffic intensity • Road geometry
(4) Vehicle	• Vehicle speed • Tire pressure • Tire structure (thread depths and patterns)

As mentioned before, this study will only focus on the road surface properties of a pavement. Because the first group (aggregate) is the only group that can be controlled by the road authorities, the other three groups will not form part of this study and will consequently not be addressed. Road authorities can hardly influence these three groups. However, it must be said that the water film thickness and the rainfall effects from group 2 can also be minimized by the road authorities by means of maintaining minimum crossfall and limiting rut depth. The vehicle speed (group 4) can also be controlled by placing traffic signs but this activity is not seen as relevant for this study.

It is very important for the road authorities to provide safety to the road users. This can be achieved by providing good visibility, proper alignment, no sharp curves or dangerous crossings but also a high level of skid resistance. This last issue is of great importance because skid resistance has great impact on the braking distance and maintaining course in adverse weather conditions. When the skid resistance of a road surface provides is low, it becomes very difficult for vehicles to decelerate or accelerate or to maintain sufficient driving stability (e.g. icy conditions in winter). It is quite clear that the deceleration is the most important issue over the course of a year. Slippery winter conditions only occur a few weeks in the year at the most. If a vehicle is not able to adequately decelerate, this may lead to undesirable consequences with traffic accidents and in the worst case to casualties.

3.1.1.1 Measurement Method Skid Resistance

Friction measurements are usually performed for two purposes. First of all, to check if the skid resistance in the initial phase of new constructed pavements is adequate enough. Secondly, to monitor how the skid resistance develops with time and to verify if the level of skid resistance still meets the requirements. The monitoring of the skid resistance is of major importance for two reasons. First of all, because according to the Dutch new Civil Code (in Dutch: 'Nieuw Burgerlijk Wetboek') road authorities are responsible for the condition of the roads which they manage. Secondly, skid resistance testing may provide data for the assessing and planning of pavement maintenance.
World wide, the standard way of measurement of skid resistance is testing on a wetted surface. This way of testing has been chosen because the deceleration on a wet surface is much lower and appears

to lead to progressively more traffic accidents than on a dry surface. A study conducted by CROW [CROW, 2005] indicates that the risk of accident on a wet road surface is five times higher than on a dry road surface. The skid resistance is measured in the near side wheel path. Although European harmonisation efforts are on their way, there is no such a thing as a standard method of skid resistance testing. Numerous procedures and devices are operational, all with their own speed, tire size, tire speed and tire inflation pressure characteristics.
The skid resistance test method commonly used in the Netherlands uses a trailer consisting of two running wheels and one separate wheel near the centre of the trailer. A smooth tire without profile is mounted to this test wheel. This wheel is powered by one of the running wheels of the trailer with a lower peripheral speed (86% retarded) than the running wheels. The static load that is exercised by the measuring wheel on the road surface is 1962 ± 9.81 N. The test is performed by getting a thin layer of water (water film) in front of the wheel. The thickness of the water film is 0.5 mm having a minimum width of 150 mm. Figure 3-2 shows an image of how the skid resistance is measured. The measuring method and equipment are specified in Test 72 of the 'Standard RAW Specifications 2010' (in Dutch: 'Standaard RAW Bepalingen 2010').

Figure 3-2: Skid resistance measurement method (KOAC-NPC).

The measured friction coefficient depends on the type of measuring equipment and the speed at which the tests are carried out. In the Netherlands, measurements are standard performed at a speed of 50 km/h or 70 km/h (see Standard RAW Specifications 2010 for details on equipment and data processing). On a regular friction testing project the Standard RAW Specifications test speed is primarily set to 70 km/h, except for the following conditions.

- in contracts where the Standard RAW Specifications of 2005 or older apply;
- in turns or exits where a speed of 70 km/h cannot be achieved.

In these conditions, the test speed is reduced to 50 km/h.

According to a study conducted by TNO [Stroefheidsmetingen: relatie tussen kale meetwaarden bij 50 en 70 km/u, 2007] on the relationship between the skid resistance measured at a speed of 50 km/h and 70 km/h, the influence of the measurement speed on the skid resistance appears to be slightly dependent on the macro texture of the wearing course. In cases of porous asphalt layers where the surface is much more rough, the effect of the measurement speed on the skid resistance is even smaller. In the referred study two equations (equations 1 and 2) are presented that can be used for converting the skid resistance measured at a speed of 70 km/h to values that would have been measured at a speed of 50 km/h or vice versa.

- For porous asphalt layer (PA): $SR(50) = 45/42 \cdot SR(70)$ *(equation 1)*
- For dense asphalt concrete layer (DAC): $SR(50) = 44/39 \cdot SR(70)$ *(equation 2)*

3.1.1.2 Limit Values Skid Resistance

An extensive study carried out by SWOV and Rijkswaterstaat resulted into a relationship between the friction coefficient measured according to the Dutch standard method (RAW 2010 test 72) and the number of traffic accidents. Based on this relationship, limit values for the friction coefficient were developed. This relationship is illustrated in a graph (figure 3-21) presented in sub-section 3.3.1. This relationship will be used later to link the data of skid resistance to traffic safety. The limit values are applicable for the test method specified in the Standard RAW Specifications, where a measurement with a 86% retarded wheel is used. These limit values are different for each type of wearing course. Distinction is made between porous, open graded asphalt mixes and dense asphalt concrete. The difference between the limit values for porous and dense asphalt concrete is due the fact that for each type of wearing course different correction factors must be applied (raw data must first be processed by using correction factors). The limit values for the skid resistance per type of surface are shown in table 3-2. These values are not valid for road surfaces treated by seal coats, chip seals, etc. No generally acceptable intervention levels are available for these types of pavement surfaces. The big shift in maintenance level minimum values between RAW 2005 and 2010 is due to the fact that different correction factors must be used (RAW 2005 → 'measuring wheel 1978' and RAW 2010 → 'measuring wheel 1998').

Table 3-2: Limit values for the skid resistance (Rijkswaterstaat).

Measurement method	RAW 2005	RAW 2010		RAW 2010	
Type of road surface	Porous/Dense	Porous		Dense	
Measurement speed	50 km/h	50 km/h	70 km/h	50 km/h	70 km/h
Warning level	0.45	0.54	0.50	0.53	0.47
Maintenance level minimum value	0.38	0.45	0.42	0.44	0.39

3.1.2 Texture

As was mentioned before, skid resistance values and limit values are dependent on the type of pavement surface. The type of pavement surface can be characterised by the texture of the road surface. The texture is a parameter describing the relatively short wave geometry of the road surface. The texture of a road surface can be divided into three different groups. These are the micro, macro and mega texture. All these three groups express specific longitudinal irregularities and are classified by different wave lengths. Irregularities with wave lengths of 0.5 mm or less are classified as micro texture. Irregularities with wave lengths between 0.5 and 50 mm belong to the group macro texture, whereas, irregularities with wave lengths between 50 and 500 mm belong to the group mega texture. Longer waves can be termed roughness or even slopes. The specifications of the longitudinal surface properties are given in table 3-3.

Table 3-3: Specifications of longitudinal surface properties.

Longitudinal surface properties	Wave length	Wave number (number of cycles per metre)
Micro texture	< 0.5 mm	> 2000
Macro texture	0.5 – 50 mm	20 – 2000
Mega texture	50 – 500 mm	2 – 20
Roughness	> 500 mm	< 2

The micro texture is determined by the angularity (sharpness) and asperity of the surface of the mineral aggregates. The micro texture affects the adhesion between the vehicle tires and the road surface. This adhesion in turn affects the breaking distance at low speed. Sufficient micro texture is required to ensure removal of waterfilm on the mineral aggregates where high contact pressures are present between the aggregates and the vehicle tire. If the micro texture cannot overtop the waterfilm on the mineral aggregates (figure 3-3), negative consequences for the traffic safety may prevail as a result of this deficiency. At lower speeds, the micro texture is the most important texture characteristic for the skid resistance.

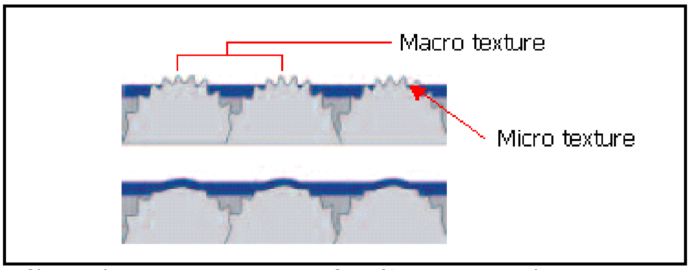


Figure 3-3: Water film on the stone aggregates surface (Source M. van de Ven presentation, Aggregates).

At higher speeds, the macro texture becomes more relevant for the skid resistance. The macro texture is determined by the particle size of the mineral aggregates at the road surface. This has an effect on two issues. First of all, it affects the deformation of the vehicle tires. Secondly, it has an effect on the drainage of water. The remaining water on the road affects the contact between the vehicle tires and the road surface. The macro texture is primary responsible for the amount of water that is stored between the stone particles (water that remains on the road). This may lead to splash and spray which in turn will have a negative effect on the traffic safety.

Surface irregularities with wave lengths between 50 and 500 mm are classified as mega texture. Mega texture may also influence the interaction between the road surface and the vehicle tires, but the mega texture have no effects on the skid resistance. It has only effect on the rolling resistance. This effect will be described in sub-section 3.2.2.1.
Another class of surface irregularity is roughness. Roughness is denoted as longitudinal irregularities with wave lengths of more than 500 mm (longitudinal unevenness of a pavement structure).

Pavement texture has not only an impact on friction issues, but on many driving aspects, such as rolling resistance, drainage, traffic noise, splash and spray. Each of these aspects is influenced by a certain range of texture (scale). The texture can have either a positive or negative effect on these aspects. Figure 3-4 illustrates the texture definitions and their influence on pavement surface properties. This figure also shows that the skid resistance is predominantly determined by the micro and macro texture. The rolling resistance is determined by the macro texture, mega texture and the roughness (unevenness) of the pavement surface. The figure also demonstrates that an enhancement of the skid resistance will have a negative effect on the rolling resistance.

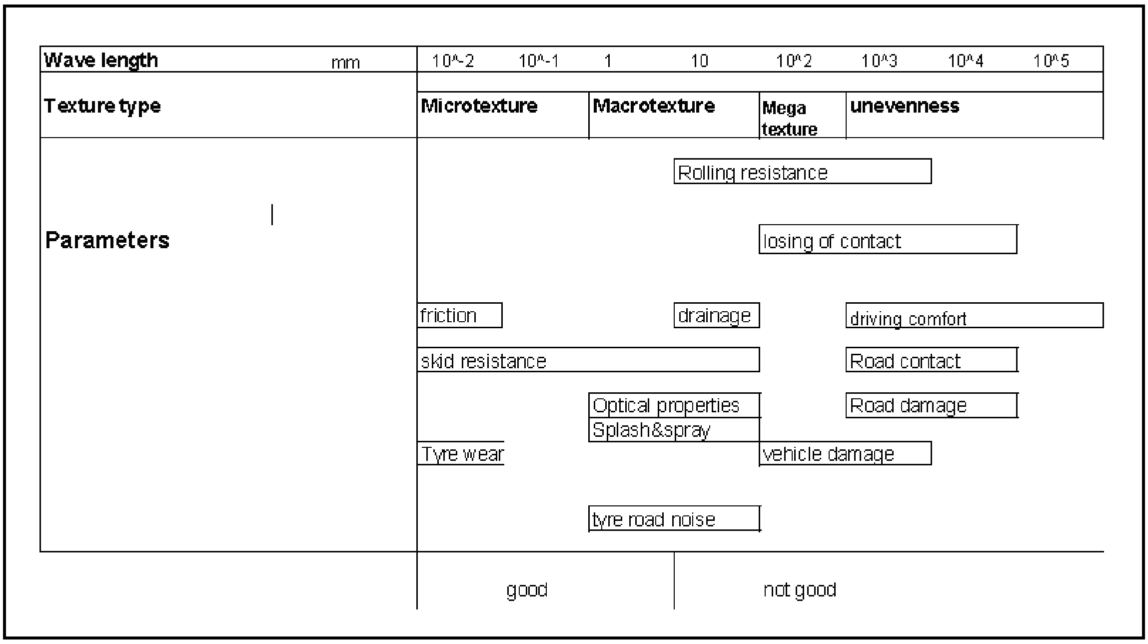


Figure 3-4: Texture definitions and their effect on road surface properties (Source M. van de Ven presentation, Aggregates).

The mineral aggregate at the top of the surface layer contains the most important aspects which allow the road surface to meet a couple of requirements. The skid resistance, texture and even water storage are dependent on the shape of the mineral aggregate and the inter-particle space. The larger this space is, the more water can be stored between the individual particles. This has an effect on how much water has to be squeezed between the tire and the road surface. The squeezing speed needs to be high otherwise the phenomenon aquaplaning may occur.

3.1.2.1 Measurement Method Texture

Measurements of pavement texture in the Netherlands are carried out with the use of the Automatic Road Analyser (ARAN) or similar devices with non-contact test systems consisting of lasers and accelerometers. The ARAN (Trademark of Fugro) is equipped with very fast lasers making it possible to scan the road surface at traffic speed. The Root Mean Square (RMS) and the Mean Profile Depth

(MPD) are the two parameters that are in international standards used for expressing the texture. The RMS value is a value that indicates the mean asperity height of a road surface. The MPD is the average value of the profile depth over a certain distance (baseline). This MPD value indicates the water storage capacity of the upper part of the road surface. The indication of the water storage capacity is more applicable to dense asphaltic concrete (DAC) and stone mastic (SMA) wearing courses and not so much to porous asphalt (PA) wearing courses. The reason for this inadequacy is because in porous asphalt layers the water easily flows through the voids between the aggregate particles of the whole asphalt layer. The MPD measured at normal traffic speeds is calculated at 100 mm intervals, although various types of equipment may use other intervals. The average of the MPD values is commonly calculated per metre road section. Measurement of texture, filtering of the raw data and computation of RMS and MPD are specified in ISO 13473. The MPD value is calculated according to the following equation (see also figure 3-5):

$$\textbf{MPD} = (1^{st}\ \textbf{peak level} + 2^{nd}\ \textbf{peak level}) / 2 - \textbf{average value} \qquad \textit{(equation 3)}$$

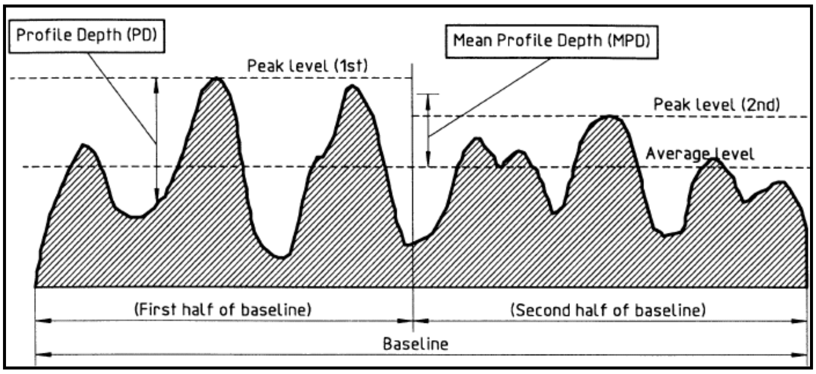


Figure 3-5: The principle to calculate the Mean Profile Depth (KOAC-NPC).

Another measure for the mean texture depth (MTD) can be obtained by the simple sand patch method (see figure 3-6). Sand Patch Method is a measure of the surface macrostructure (Sand Patch Texture Depth). This is a stationary measurement where traffic measures are needed. This test is very slow and labour intensive for providing only one single value per test point.

Figure 3-6: Sand patch measurement method (Source: highwaysmaintenance.com).

3.1.2.2 Water Permeability of Porous Asphalt Layers

Porous asphalt layers are applied because they are an effective measure for traffic noise abatement and also for reducing splash and spray water. The permeability of wearing courses is a very important factor to traffic safety in rainy conditions. The measurement of the water permeability of porous asphalt wearing courses (PA, in Dutch ZOAB) is usually determined by using a drainometer also called the Becker device (figure 3-7). This is a cylindrical tube with a round outflow opening at the bottom which is placed at the porous asphalt layer under investigation. The cylindrical tube is loaded with an extra weight so that the porosity of the surface texture is closed by the soft rubber under the perimeter (base) of the tube. The tube is filled with water and the time between the moments when the water level passes the upper and lower mark is measured. This outflow time is the final result of the test. The smaller the outflow time, the larger the permeability of the wearing course. It must be noted that the outflow time can vary significantly over a road section. Disadvantages of the drainometer is the difficulty of the closure around the opening at the bottom of the tube and the relative large amount of water that is needed.

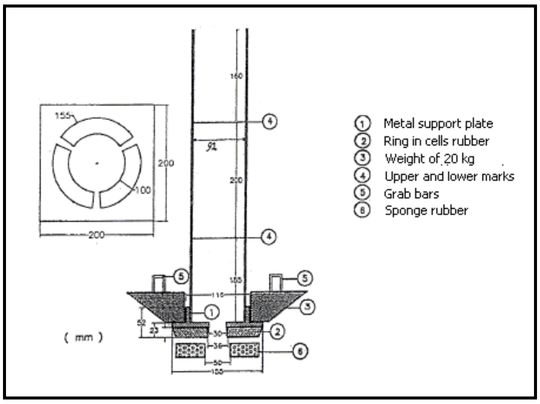


Figure 3-7: Becker device to measure water permeability of wearing courses in situ.

3.1.2.3 Limit Values Water Permeability for Porous Asphalt (PA, ZOAB)

- For new porous asphalt wearing courses the outflow time must be 25 seconds or less. No distinction is made between main, provincial and urban roads.
- Cleaning is required if the outflow time results in values between 50 and 75 seconds. After cleaning the outflow time must be 30 seconds or shorter.
- For outflow times of more than 75 seconds, no means are generally available to clean the pavement or to improve permeability anymore in order to attain an outflow time of 30 seconds or less.

3.1.3 Roughness

Surface roughness is the distress phenomenon of longitudinal irregularities with wave lengths of more than 500 mm. Surface roughness affects the motion and operation of a moving vehicle through its effect on the users perception of ride quality. Besides that, road roughness also has an effect on the traffic safety and fuel consumption by the vehicles. This will be further discussed in the sub section 'Traffic safety' and 'Fuel consumption'. The initial roughness of a new pavement is affected by construction quality. Road surface roughness can develop due to increasing cracking and a reduced load distribution in the pavement structure (unequal compaction quality). The primary cause is usually found in unequal settlements of the subgrade. Figure 3-8 and 3-9 illustrate a long and short wave road unevenness. Short wave unevenness, termed as corrugations can often be seen at traffic lights. Corrugations are due to insufficient stability of the asphalt mixes in the upper two layers of an asphalt pavement. The waves lengths vary usually around 500 mm or even shorter.

Figure 3-8 : Long wave road unevenness (left).
Figure 3-9: Short wave road unevenness 'corrugations' (right).

3.1.3.1 Measurement Method Roughness

The measurement of the longitudinal profile can be carried out with different measurement systems. In the Netherlands, the HSRP (High Speed Road Profiler) and the ARAN are the two most used measurement systems. Both measurement systems consist of a laser, accelerometer and an odometer. The laser provides a quick scan of the road surface and measures the distance between the road surface and the unit in which the laser is mounted. The accelerometer measures the vertical acceleration of the moving vehicle. The accelerometer data are used to convert the moving plane of the laser to a stable plane in which vertical movement of the test vehicle has been filtered. The data of the travelled longitudinal distance is provided by the odometer.

The longitudinal profile of a road is measured in the near side wheel path (the right-hand wheel path in the Netherlands) and between the wheel paths. The measurements result in a longitudinal profile in the wavelength range between 0.6 and 90 m. The test speed varies usually between 40 and 90 km/h. The slower the test vehicle drives, the more the wave length interval shifts to shorter wave lengths. In the Netherlands, the standard test speed commonly is 80 km/h. If during routine testing the speed varies too much or drops below 40 km/h, the measured values are declared invalid. The sum of the vertical displacements of the vehicle per unit length indicates the roughness parameter. This parameter is the IRI (International Roughness Index) and is expressed in m/km. Other parameters can be derived from the collected raw data. The raw data allow simulation of various types of roughness testing on the basis of the profile data.

3.1.3.2 Limit Values Surface Roughness

In the Netherlands the limit value for road surface roughness on motorways is 3.5 m/km. Table 3-4 shows the rating scale of roughness on Dutch motorways.

Table 3-4: Rating scale of roughness on Dutch motorways (CROW, 2005).

IRI [m/km]	Qualification	Notes
< 0.6	Good	virtually no roughness
0.7 – 2.0	Sufficient	minor roughness
2.1 – 3.4	Moderate	rough pavement; planning and budget for maintenance
3.5 – 4.9	Insufficient	maintenance is required in short term
> 5.0	Poor	overdue maintenance; maintenance is directly required

3.1.4 Rutting

Rutting can be described as the permanent deformation in the transverse profile of a road pavement, caused by the repeated heavy traffic loads (especially heavy trucks) usually during hot weather days. Ruts can be formed through the deformation of the asphalt concrete pavement or by distortions in the underlying layers (base, sub-base and subgrade). The rutting depends also to some extent on the type of wearing course. Deformation in the asphalt layers is defined as primary rutting. Deformation in the underlying layers (foundation) is referred to as secondary rutting. This last type of rutting is found in situations with problems with the load carrying capacity. From all the rutting cases, 90% of the cases is primary rutting. Rutting is a safety problem for road users for adequate manoeuvring of their vehicles on the road. This problem may get worse when water accumulates in the ruts. This water storage can contribute to the phenomenon of hydroplaning or aquaplaning. Aquaplaning occurs when the tires of a driving vehicle loose contact with the road surface due to the layer of water lifting the vehicle tires from the road surface. This leads to loss of friction between the tires and the road surface and the dangerous situation where the driver has no control over the vehicle anymore. Most of the time this event leads to accidents.

Another disadvantage is that large puddles are formed in the ruts. This leads to the problem of splash and spray water. The effects of rutting on the traffic safety can be rated as negative since the vehicle control and the visibility for the road users can be reduced. Severe rutting can impede steering for a road user to steer his vehicle out of the rut. In addition to this, severe ruts have sometimes even a positive effect as these ruts help the driver to manoeuvre the vehicle on the road. In other cases, deeper ruts have also another positive effect because drivers are then forced to reduce speed in order to keep their vehicles under control. The last two mentioned positive effects have as a result that the number of traffic accidents will decrease.

3.1.4.1 Measurement Method Rut Depth

Rutting can be 'visually' measured or with the help of the ARAN or similar devices. The ARAN measures the rut depth accurately by using lasers. The measurements are commonly performed each 5 m of travelled distance. The 'visual' measurement of the rut depth and water layer depth is done by means of a 1,2 m long straightedge (see figure 3-10).

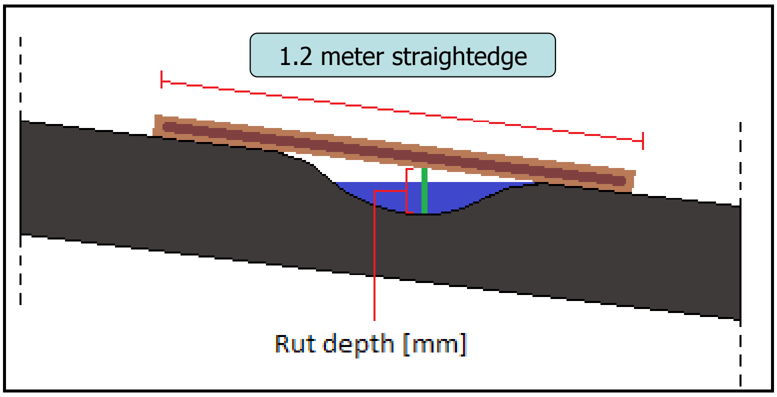


Figure 3-10: Rut depth measurements under a steel rule of 1.2 m.

3.1.4.2 Limit Values Rut Depth

According to the Dutch guidelines on main roads (Rijkswaterstaat) maintenance is required when the rut depth reaches a value of 18 mm or more. The warning limit value is 16 mm. Table 3-5 shows the rating scale of rut depth on Dutch motorways.

Table 3-5: Rating scale of rut depth on Dutch motorways (CROW, 2005).

Rut depth [mm]	Qualification	Notes
< 5	Good	virtually no rutting
5.0 – 11	Sufficient	minor rutting
12 – 17	Moderate	planning and budget for maintenance
18 – 23	Insufficient	maintenance is required in short term
> 24	Poor	overdue maintenance; maintenance is directly required

3.1.5 Crossfall

In the Netherlands asphalt pavements are designed with a minimum crossfall of 2.5%. This crossfall is provided to ensure good surface drainage of rainwater. The water can flow easily away from the road surface to the side of the pavement. Crossfalls are also applied in curves through super elevation to partly compensate the centrifugal force. In addition to this it also has benefits for the ride comfort of the road users in curves.

In cases of rutting, the crossfall of a pavement plays an important role on the risk of an amount of water to be stored in the rut. By providing all roads with a sufficient level of crossfall the water layer depth in the ruts can be lowered. Figure 3-11 demonstrates what the difference is with the water layer depth in the rut between a pavement without and with crossfall. As was mentioned in the previously paragraph, the accumulated water in the rut may lead to aquaplaning and the problem of splash and spray water.

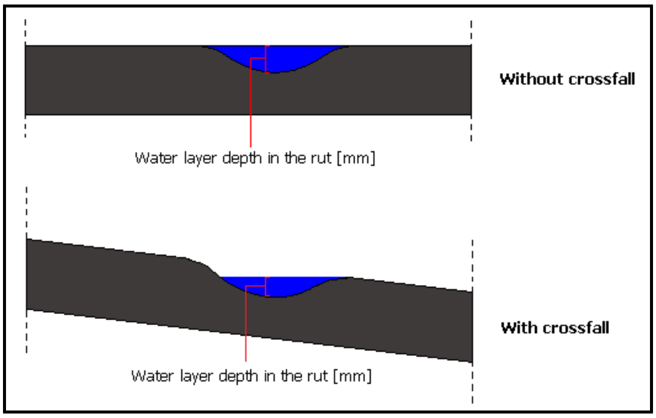


Figure 3-11: Difference between the water layer depth in a rut on a pavement with and without crossfall.

3.1.5.1 Limit Values Crossfall

In the Netherlands, a crossfall between 1% and 5% is permitted. However, in curves larger slopes can be and are applied. The crossfall is generally limited to a maximum of 5% to avoid sliding of vehicles in the transverse direction during winter periods with icy roads.

3.2 Environmental/Sustainability Aspects

3.2.1 Traffic Safety

Traffic safety is the most important requirement for both the road users and road authorities. Most of traffic accidents are caused due to human mistake. But sometimes the road surface also plays a role in an accident. In principle three components may play a role in the cause of any accident. The three components that may contribute to the cause of a road accident can be either the driver, the road, or the vehicle or the road surroundings. Approximately 90% of the accidents on the Dutch road network are caused by human mistake. The other 10% are either caused due to failure of the vehicle or the pavement structure itself. In a few cases, the cause of the accident can be attributed to a combination of two or even three of these components. In Belgium it was found that the pavement structure is responsible for approximately 25% (34/140) of the total number of accidents (OCW, De weg: actor van duurzame mobiliteit). The involvement of the three components in road accidents in Belgium is illustrated in figure 3-12.

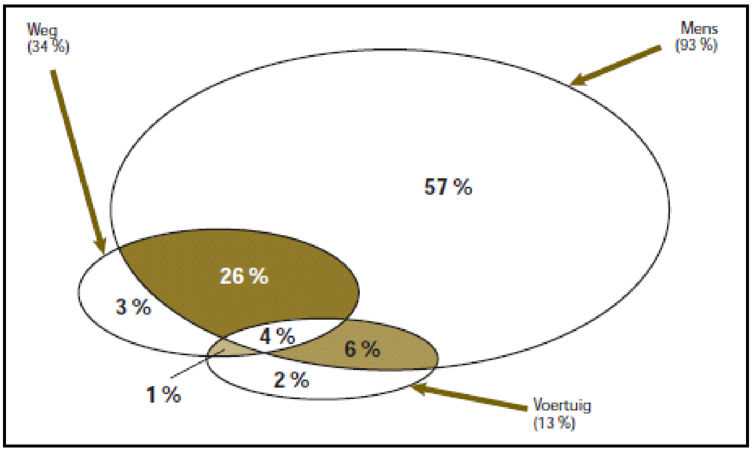


Figure 3-12: Involvement of the three components in road accidents [%] (Source OCW).

With the 'pavement structure' is referred to the road surface properties because these are the main characteristics that may play a significant role in case of an accident. Therefore this study will mainly focus on the road surface properties of pavement structures rather than the total pavement structure.

3.2.1.1 Objective and Subjective Safety

Traffic safety can be differentiated in objective and subjective safety. The objective safety is measureable and is presented for example in the number of traffic accidents (fatal accidents). The subjective safety refers to individual safety experience and the individual response to this. In practice, people turn out to manage their own safety level where risk compensation takes place. Short following distance is an example of this. Another example are the higher speeds when raining on porous wearing courses where no or at least less splash and spray is encountered compared to dense pavement surfaces. The expected safety improvement evaporates simply because of the higher speed. In the assessment of the traffic safety the subjective safety plays an important role. A road user can anticipate observable issues or known situations in front of him but not unnoticeable or unexpected factors such as a reduced skid resistance or sudden discontinuities on a perfectly smooth road. For roads with a speed limit of 60 km/h skid resistance, texture, and longitudinal evenness are the most important properties for the assessment of the safety of road surfaces. From these properties, the skid resistance and texture are the not-noticeable properties. Local unevenness such as potholes, ridge formation or height differences at connections to bridges will form discontinuities, which will lead to dangerous situation for the road users.

Each year, many accidents do occur on the Dutch road network, specially on provincial roads. According to a study performed by CROW [CROW 2005], most of the fatal accidents occur on the provincial roads where a speed of 80 km/h is allowed (figure 3-13). This figure shows the distribution of the total number of fatal accidents (single vehicle) between 1993 and 2002 over various allowed travelling speeds. It also illustrates at the same time the difference of the total number of fatal accidents on local (50 km/h), provincial (80 km/h) and national roads (100 km/h). The results of this study led to the decision to focus in this study mainly on the situation of the arterial and provincial roads of the Dutch road network.

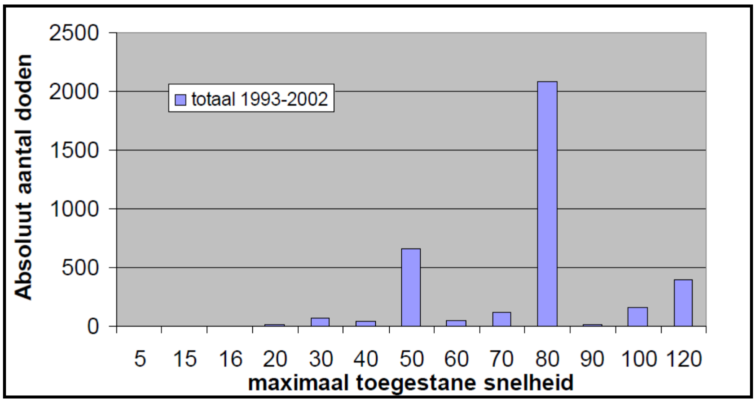


Figure 3-13: Number of fatal accidents in single vehicle accidents between 1993 and 2002 divided in speed regime.

3.2.1.2 Crash Rate

The crash rate is determined by many pavement related factors. The most important factor is the braking distance. Road users should be able to decelerate within a short distance in case of emergency. In the Netherlands, the minimum required braking deceleration is 5.2 m/s^2.
The braking distance is influenced by the skid resistance of a road surface. The other relevant factors are displayed in the yellow box in figure 3-14. This figure indicates also the properties of a road surface which have influence on the mentioned factors that influence the crash rate. The last column of figure 3-14 lists the expressions given for the road surface properties except for the factor 'visibility'.

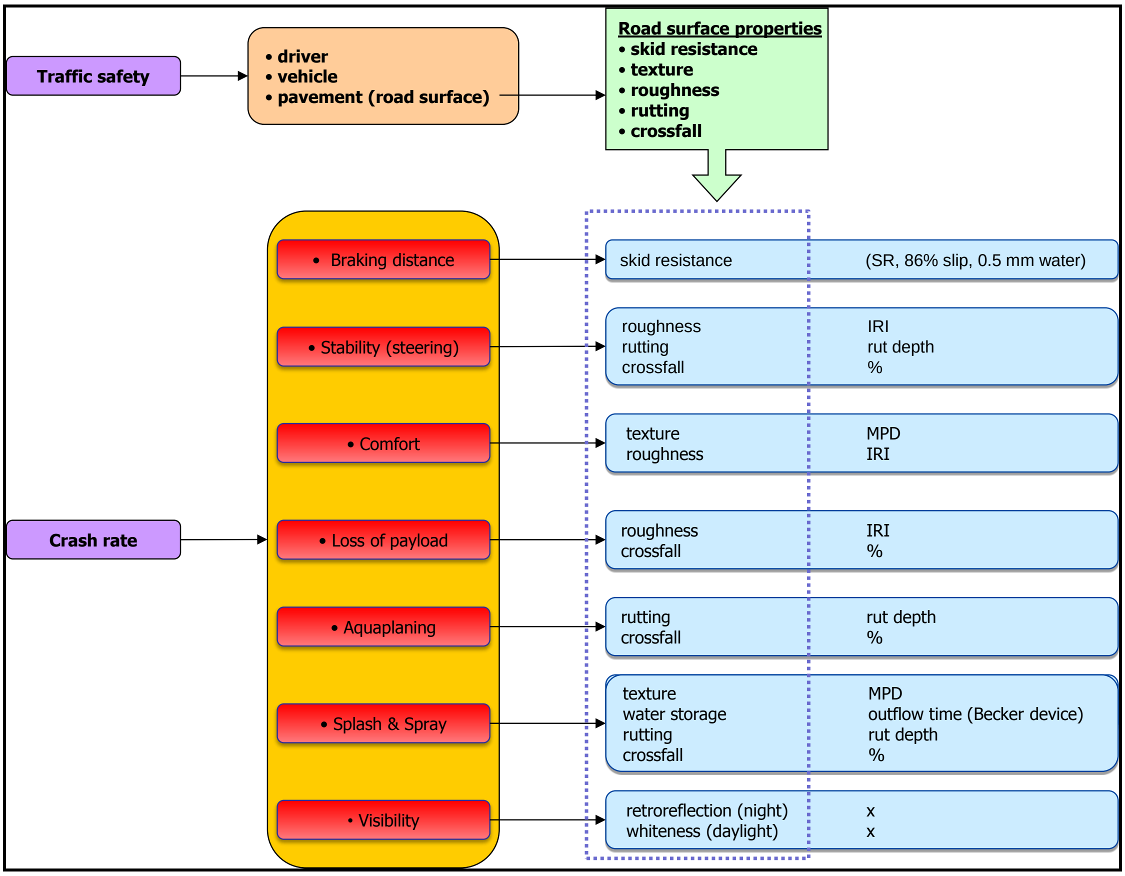


Figure 3-14: Overview of the factors (yellow box) which determine the crash rate in relation with the properties of a road surface which have influence on the previously mentioned factors.

For the factor 'visibility' road surface markings play a significant role in case of traffic safety. Road surface markings are used to provide guidance and information to the road users. A road without markings can lead to unsafe situations. The markings provide the road users information on how to use the road. They indicate the borders of a road and divide the carriageway in traffic lanes. The horizontal stripes make it possible to let road users evaluate their relative position on the road and to evaluate the distance from other vehicles in front of them. In fact the markings on the road surface allow for communication between the road surface and the road users. During daylight the luminance (whiteness) of the markings is important whereas during the night the retroreflection of the markings is of more importance. The luminance and retroreflection of road surface markings are actually not properties of the road surface itself. For this reason the factor visibility is neglected in this research.

3.2.1.3 Observable and Non Observable Surface Properties

The non observable surface properties and the surface properties which provide a false sense of safety to the road users are the most important factors of road surfaces. For example road users used to drive faster when they know they are driving on a porous asphalt layer. In contrast, they will drive slower in adverse weather conditions when they are aware that they are driving on a dense asphalt concrete. This clearly shows that road users adapt their driving behaviour to the local situation. So in practice it turns out that road users manage their own safety level.

3.2.2 Fuel Consumption

The environmental impact of highway projects is nowadays a very important aspect that has to be taken into consideration for achieving a sustainable development and environment. Experience from the last few years reveals that more and more attention is paid to this topic. As a result of this, energy consumption also became a very important aspect in the planning process for highway projects. Subsequently, more attention is paid on the fuel consumption by the passing traffic throughout the entire pavement life of these road pavements. According to a study performed by EAPA (European Asphalt Pavement Association), the energy consumption during the construction and maintenance phases over a period of 30 years is 10 to 345 times less than the energy consumed by the traffic during the operational phase of a road. Figure 3-15 presents the energy consumption for different types of pavement structures, compared with the energy consumption of the total cumulative traffic. From this figure can be concluded that fuel consumption is a very important environmental aspect for road pavements.

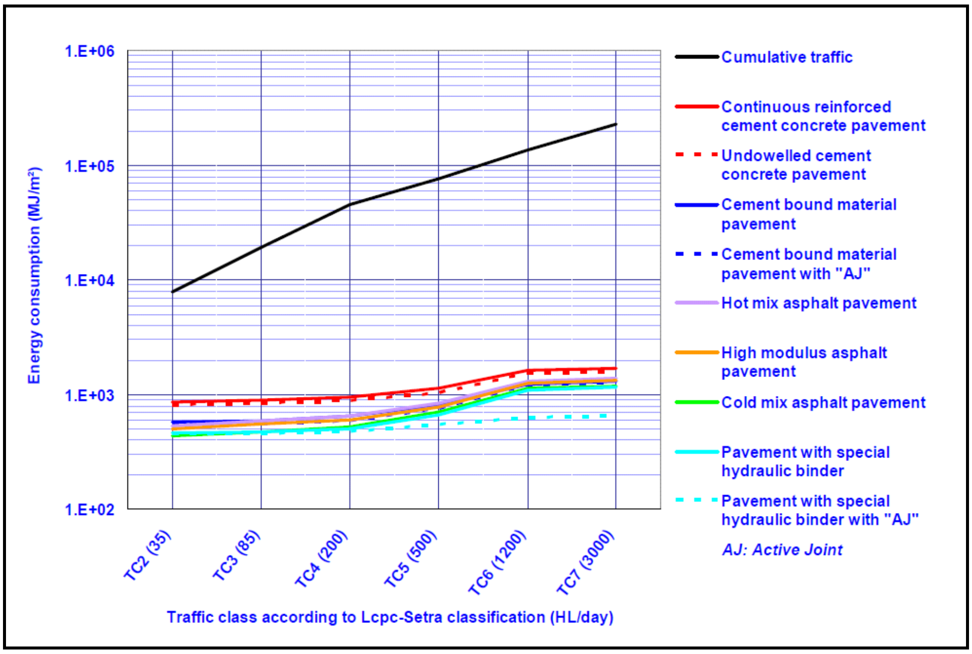


Figure 3-15: Energy consumption for different types of pavement structures compared with the energy consumption of the total cumulative traffic.

One of the main principles for achieving a sustainable development is reduction of CO_2 emissions. The ambition of both the government and road authorities is reduction of greenhouse gases emitted by the road traffic. There are many indications that huge savings can be achieved on the fuel consumption by vehicles. In these times it has been very important for all types of vehicles to be more fuel economic. This means that nowadays new vehicles should consume less fuel in comparison with the vehicles from the past. Less fuel consumption means also less CO_2 emissions. Many studies and efforts have already been completed by vehicle manufacturers to make their vehicles as fuel economic as possible. But in contrast to the work done by the car manufacturers, not so much work has been done in the field of pavement structures for reducing the fuel consumption of all types of road traffic. It can be expected that improvements of the road surface properties will also lead to significance reductions in the fuel consumption by vehicles. This has already been confirmed by different studies on this topic. Some of these studies have demonstrated that improvement of the road surface rolling resistance will have a huge effect on the fuel consumption of cars and trucks. The lower the rolling resistance, the less fuel will be consumed. Energy consumption significantly depends on the weight of the vehicle.

There are many other factors that may have an impact on the fuel consumption by vehicles. Two of these factors that are clearly dependent on the surface, are the longitudinal gradient resistance and the rolling resistance. The longitudinal gradient resistance is caused due the slopes of a road especially in rolling or mountainous terrain. This gradient is not further elaborated in this study. The crossfall was discussed earlier in subsection 2.1 *Road surface properties*. The rolling resistance will be discussed in the next subsection. The rolling resistance is seen as the most indicative parameter for fuel consumption in this study. This is especially the case for a country like the Netherlands (flat terrain).

3.2.2.1 Rolling Resistance

This section describes the rolling resistance and shows how this parameter is measured in practice. Further, the factors affecting the rolling resistance will be discussed. Subsequently, the effect of the rolling resistance on the fuel consumption will be clearly identified. At the end of this paragraph a list of models is given that can be used to determine the rolling resistance of road surfaces.

Rolling resistance is defined as the friction that occurs when a round object rolls on a surface. It can also be defined as the force that is caused due to the interaction between the tire of a vehicle and the road surface. This force acts in the opposite direction to the travel direction. The force is determined for a major part by the texture of the road surface, specifically the macro and mega texture. This can also be seen in figure 3-4. The rolling resistance force is nearly proportional to the normal force acting on the road surface. This quotient (rolling resistance force / normal force) is called the rolling resistance coefficient (RRC) (see equation 4).

$$\mathbf{F_{rolling\ resistance}\ [N] / F_{normal\ force}\ [N] = RRC} \qquad \textit{(equation 4)}$$

The rolling resistance can be determined by the energy loss in the tire, road surface and their interaction when the tire is rolling. The main reason of the energy loss is due to the deformation of the tire. The greater the deformation of the tire, the larger the contact area between the tire and the road pavement will be, which has as result a major effect on the magnitude of the rolling resistance. A list of the factors that influence the magnitude of the rolling resistance is given below.

- Macro deformations in the contact area between the tire and road surface (macro texture of the road surface);
- Friction in the contact area between the tire and the road surface (texture);
- Shape of the tire;
- Inflation pressure of the tire;
- Travel speed;
- Tire load (vehicle weight).

This study will only consider the properties of the road surface (texture and roughness) that have an influence on the rolling resistance. As a result, the factors governed by the tire or vehicle are disregarded in this study.
The rolling resistance coefficient can be seen as a property of the tire or the road surface. For simplicity in this study it will be assumed that if a low rolling resistance has been measured on a specific road surface, this value will stand for all types of tires and vehicles passing over that road surface.

The rolling resistance is responsible for a certain part of the energy consumption. According to a study performed by TNO [Giezen et al., 2012], the part of the energy which is consumed due to the rolling resistance is estimated to be between 18 and 32%. The rest of the energy is consumed due to air drag, inertia forces and mechanical friction. All these mentioned factors are influenced by the vehicle velocity. Figure 3-16 illustrates how the air drag increases quadratic with the vehicle velocity, while the rolling resistance hardly increases. This means that the faster the vehicle travels, the less the proportional contribution of the pavement surface to the fuel consumption. In addition, the impact of the rolling resistance varies greatly in the total fuel consumption due to the traffic volume. In traffic jams for example, constant acceleration and deceleration take place but in this study no attention will be paid to the effect of the traffic volume on the fuel consumption.

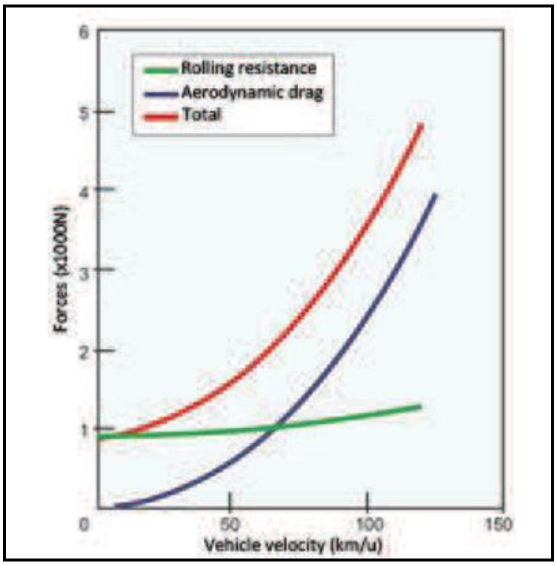


Figure 3-16: Vehicle velocity in relation with the rolling resistance and air drag.

In this study the choice has been made to look only at the two road surface properties texture and roughness, although unevenness in the transverse direction of a pavement due to rutting and crossfall may also influence the rolling resistance. Since the influence of rutting and crossfall in comparison with the influence by the texture and roughness is expected to be very small, only the previously mentioned two properties texture (MPD) and roughness (IRI) will be analysed. Figure 3-17 displays this choice graphically.

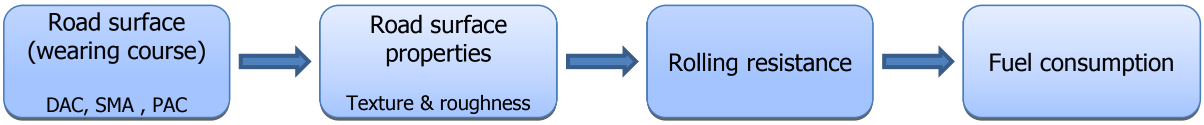


Figure 3-17: Graphic display of research with regard to fuel consumption.

On the basis of the types of wearing courses a distinction is made between positive and negative texture. Both textures have some influence on the interaction between the tires and the road surface. A positive texture applies to a road surface where the surface is formed by upwards projecting aggregates. A negative texture is formed by cavities in the flat surface (see figure 3-18). Dense Asphalt Concrete and Stone Mastic Asphalt are typical examples of a positive texture in which the texture becomes more positive during the pavement life. Porous Asphalt Concrete has a negative texture. A negative texture will have a more favourable effect on the rolling resistance (i.e. will reduce the rolling resistance), whereas a positive texture has a more unfavourable effect on the rolling resistance. A possible explanation for this can be the larger deformation of the tire. This occurs because the positive texture of the asphalt penetrates into the tire. The Mean Profile Depth (MPD) can be used as a measure for both the negative and positive texture. Currently no standard measurement method is readily available to discriminate between a positive or a negative texture.

Skewness: measure for the asymmetry of the texture (figure 3-18)

- Positive skew: wide descend, sharp tops
- Negative skew: narrow deep descend valleys, flat tops

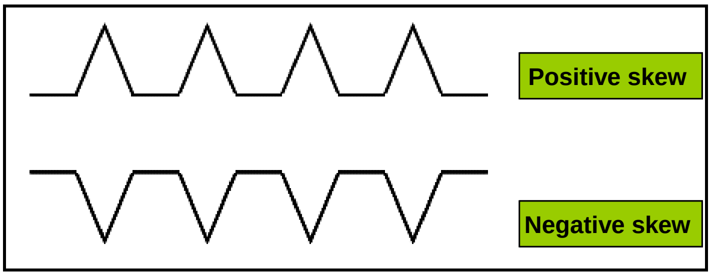


Figure 3-18: Difference between positive and negative skew.

The rolling resistance that vehicles experience when travelling over a road is highly dependent on the type of road surface. In case a vehicle drives from a favourable to an unfavourable road surface, the rolling resistance can increase up to a factor two. The rolling resistance coefficients for different types of wearing courses are shown in the figures 3-19 and 3-20. These figures are extracted from a research project carried out by Rijkswaterstaat in the context of IPL and IPG (Innovatie Programma's Luchtkwaliteit en Geluid). The data is based on measurements performed on the test section in Kloosterzande. The differences found in the rolling resistance coefficient can lead to a potential saving on the fuel consumption by choosing for the correct type of wearing course. The figure clearly shows that out of the three types of wearing courses PAC, SMA and DAC, the porous asphalt layer has the highest rolling resistance coefficient and the DAC layer the lowest. A possible explanation for the lower rolling resistance coefficient for the SMA and DAC in comparison with PAC can be due to larger deformation of the tire in the spaces between the mineral aggregates (larger distance between flat tops of a negative texture).

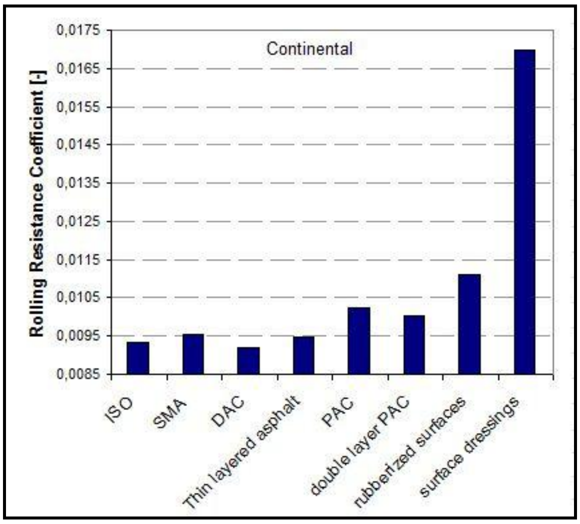


Figure 3-19: Rolling resistance coefficient for different types of wearing courses.

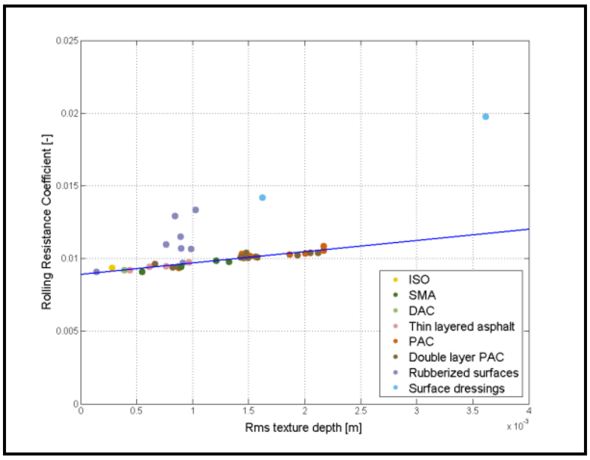


Figure 3-20: Root Mean Square (RMS = average texture roughness) vs. rolling resistance coefficient.

3.2.3 Traffic Noise

Traffic noise is often an annoyance for people that live in the surroundings of heavily trafficked roads. Daily noise nuisance may be harmful to their health in some cases. Despite all the efforts of recent years to reduce traffic noise in the Netherlands, the traffic noise remains still one of the most common problems related to the road transport system. Due to the growth of road traffic it is expected that this problem will become even larger. The current standard measures for noise abatement focus more and more on improvement of social living conditions leading to a dramatic rise in costs of these measures. So there is an urgent need for alternatives that are more cost effective. The idea is not only to come with measures that prevent noise nuisance but to tackle the problem of traffic noise at the source of noise production. The next paragraph will touch on the sources of traffic noise in order to get more knowledge of the background of traffic noise.

Traffic noise can be interpreted as the noise emitted from vehicle engines and exhausts and the noise they produce by the interaction with the pavement. The noise emitted from engines and exhausts is also called the operating noise. The noise generated by the interaction between vehicle tires and the road surface is called the 'rolling noise'. For passenger cars and trucks, the rolling noise is dominant over the operating noise for vehicle speeds of more than 30 km/h and 60 km/h, respectively. The noise emitted from the engines and the interaction between the tire and the pavement can be attributed to several source factors. These are the pavement factors, the tire factors and the vehicle factors. The pavement factors responsible for noise generation are the following:

- Pavement type (dense or open graded asphalt);
- Surface texture (macro and mega texture, macro especially for the in-vehicle noise);
- Shape, dimensions and physical properties of the aggregates;
- Temperature.

The tire and engine factors responsible for noise generation are the following:

- Tire properties (thread type, speed and temperature);
- Vehicle speed (as speed increase, tire noise also increases);
- Vehicle type (passenger car or truck);
- Driving conditions (wet pavements increase pavement noise).

It must be said that these factors are interrelated and have an impact on the overall sound level. The major contributor to the overall sound level is the tire-pavement noise. This is determined by the pavement type and pavement surface texture. Dense graded asphalt surfaces generate more noise by air pumping because air is more easily trapped. Open graded asphalt surfaces reduce the air trapping consequently reducing the resulting generated noise. As an alternative to reduces traffic noise is the application of porous asphalt (ZOAB) as wearing course. According to a study performed by VBW [VBW - Asphalt, 1986], open graded asphalt pavements appear to initially produce a benefit of 3 dB(A) reduction in noise levels. Porous asphalt reduces the air trapping. However, the noise reduction benefit is lost when the voids become clogged and the aggregates at the pavement surface get polished. Raveling is another phenomenon that can reduce the benefit of noise reduction of porous asphalt. Periodic cleaning (preventive maintenance) can be performed to avoid clogging of the air voids, which prolongs quieter properties of the pavement.

In this study the factors 'surface texture' and 'pavement type' from the previously mentioned factors responsible for noise generation will be addressed. These are the only two factors that can be categorized as properties of the road surface. No attention will be paid to the properties of the aggregates because in the end these are the properties (dimensions of applied aggregates) that determine the texture depth and wavelength. The smaller the particle size of the aggregate the smaller the texture depth will be.

As mentioned before, the tire-pavement noise is responsible for a great share of the overall traffic noise level. From acoustic considerations it appears that both the depth and the wavelength of the surface texture have a considerable effect on the noise production. As can be seen from figure 3-4, macro texture has a significant impact on noise generation. This involves a direct relationship between traffic safety and noise control because the macro texture is the primary concern for moving water to maintain tire contact with the pavement surface.

3.2.4 Durability of Wearing Courses

A durable wearing course is a wearing course that performs for a long time above the acceptance level. By doing so, less maintenance will be required. Less maintenance also means less costs. For many years road authorities and their operational units are challenged to manage and maintain their roads for the road users. Durable road surfaces or long life pavement surfaces are seen as particularly desirable on all types of road especially on major roads to avoid the costs of road maintenance works. Reducing the number of times that road maintenance works have to be carried also reduces delays that the maintenance works inflict on road users on heavily trafficked roads.

By seeking for durable road surfaces or low maintenance pavements one can at the same time strive for safer and comfortable road surfaces. A good and a safe road has to be flat and should not exhibit unevenness or potholes. Unevenness or potholes on the roads do increase the chances for road accidents and make driving more tiring for the road users. The risk of losing payloads also becomes larger. Besides the positive contribution of an even road to the ride comfort, it also contributes to other aspects. These aspects are the service life of a pavement, the traffic safety and the reduction of traffic noise and vibrations.

In the Netherlands, approximately one billion Euros is being spent each year for managing and maintaining the major highway network. By spending just a slightly higher total amount, the skid resistance and the roughness of the roads for example can be improved considerably. Due to these improvements, this extra budget will result in safer roads and a longer service life of the roads. So the extra costs will be recovered pretty soon.

The final model that will be developed in this study will also help road engineers to make the best choice when choosing mineral aggregates for the wearing course. The mineral aggregates in the wearing course should provide a stable and accepted skid resistance. They have to ensure that they create a road with durable and environment-friendly road surface characteristics and also good functional properties. Resistance to polishing and ravelling may contribute to a durable and environment-friendly road surface texture that provides a high skid resistance and reduction of traffic noise.

Two of the requirements of mineral aggregates in hot mix asphalt are that they have to be strong (not crushable) and durable (low polishing rate). With strong is meant that the aggregates have to provide sufficient strength without breaking easily apart in smaller pieces. With the durability of the mineral aggregates is meant that these aggregates need to reach the expected service life without providing a quality below the acceptance level. For example, some mineral aggregates may have at the beginning a rough surface texture, but may get polished with time due to the repeated traffic loads. These types of mineral aggregate are not adequate for a wearing course.

3.2.4.1 Polishing

For a safe road with regard to skid resistance it is important to use mineral aggregate which is resistant to polishing. The resistance to polishing is different for each type of mineral aggregate. The decrease of the skid resistance is determined for a major part by the polishing of the mineral aggregate in relationship with the traffic passed. The resistance to polishing can be determined in the laboratory, because so far no equipment is available for in situ testing. The rate of polishing in situ can be determined by periodic skid resistance measurements. This is also the way in which the polishing rate of the mineral aggregates is addressed in this study.
A measure of the skid resistance of a mineral aggregate to polishing is the Polished Stone Value (PSV) of an aggregate. The property of a mineral aggregate which gives an aggregate a high PSV is the micro texture of it. A mineral aggregate that retains a substantial micro texture after polishing is an aggregate that gives a good resistance to skidding. Such type of aggregate has also a high Polished Stone Value.

3.2.4.2 Raveling

Raveling is the damage phenomenon where mineral aggregates are lifted from the road surface. This phenomenon occurs by the combination of time (hardening of the bitumen), weather condition and repeated traffic loads. Raveling can also be defined as a modification of the road surface texture. If one mineral particle comes loose from the road surface, the adjacent particles will loose their support. If the bitumen offers not enough adhesion for the adjacent aggregates, these particles will also come off and so on and so on. Because of the repetitive nature of this mechanism the damage to the pavement structure may become extensive. So it is very important for the mineral aggregate and the bitumen to provide sufficient resistance against the wearing by the repeated traffic loads. Raveling is the governing damage mechanism in porous asphalt concrete due to the large voids contents in the asphalt mixture. Furthermore the surface texture of the aggregate used plays a huge role in developing the bond between the aggregate and the bitumen. How stronger this bond is, the smaller the risk for raveling. Raveling has many consequences for the durability of the wearing course. Besides the durability of the wearing course, raveling has also its consequences for the traffic safety, driving comfort and traffic noise.

3.2.4.3 Measurement Method Raveling

On busy roads lasers are commonly used to assess raveling. On less busy roads raveling is usually assessed visually and documented as the percentage stone loss from the total measured road surface area.

3.2.4.4 Increase of MPD

As was discussed earlier in subsection 3.1.2, the MPD indicates the average value of the profile depth over a certain distance (baseline). This average value over a certain distance can be indicated with a reference line. In figure 3-21 this reference line is illustrated with a red dotted line. If no raveling occurs the reference line will remain at shallow depth from the top of the road surface (upper two pictures). By losses of stones this reference line will drop to a lower level far deeper from the top of the surface layer (bottom two pictures). So a possible explanation for the increase of the mean profile depth of DAC and SMA wearing courses is probably due to stone loss (raveling).

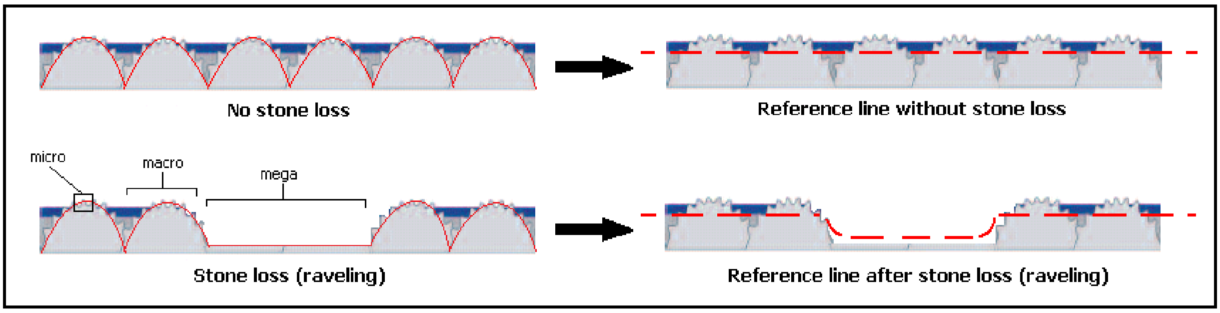


Figure 3-21: MPD (reference lines) without stone loss and after stone loss.

3.2.4.5 Decrease of MPD

As a possible explanation for the decrease of the mean profile depth of PAC wearing courses can be due to clogging of the pores or 'vetslaan' (excess of bitumen at the pavement surface). 'Vetslaan' is a phenomenon that occurs primarily in asphalt mixtures with a high bitumen content. It is usually caused by traffic load in combination with high temperatures in engorged asphalt mixtures. In addition to this there must be said that the skid resistance can also be reduced due to this phenomenon.

3.3 Relationships Between Road Surface Properties and Traffic Safety

In this part of the report the relationships between the road surface properties and the crash rate found in the literature study is addressed. Each relationship will be extensively discussed. At the end, the choice for specific relationships will be explained.

3.3.1 Skid Resistance vs. Crash Rate

A study performed by SWOV [Verkeersongevallen en wegstroefheid, 1973] revealed a relationship between the skid resistance and the number of traffic accidents. This relationship clearly shows that a decrease of skid resistance results into a greater risk of road accidents. This is illustrated in figure 3-22. The graph makes distinction between the traffic intensity and nine classes of skid resistance. Besides that, also distinction is made between two different road types: main motorways and other national roads. This study also reveals that there is an increasing chance of involvement with increasing traffic intensity. The crash rate (also termed accident quotient/ratio) is expressed in the number of accidents per million vehicle-kilometres (per mio vhc-km). The skid resistance values associated with the nine classes of skid resistance are given in table 3-6. The reported study formed the basis on which the Dutch standards for skid resistance established in 2004.

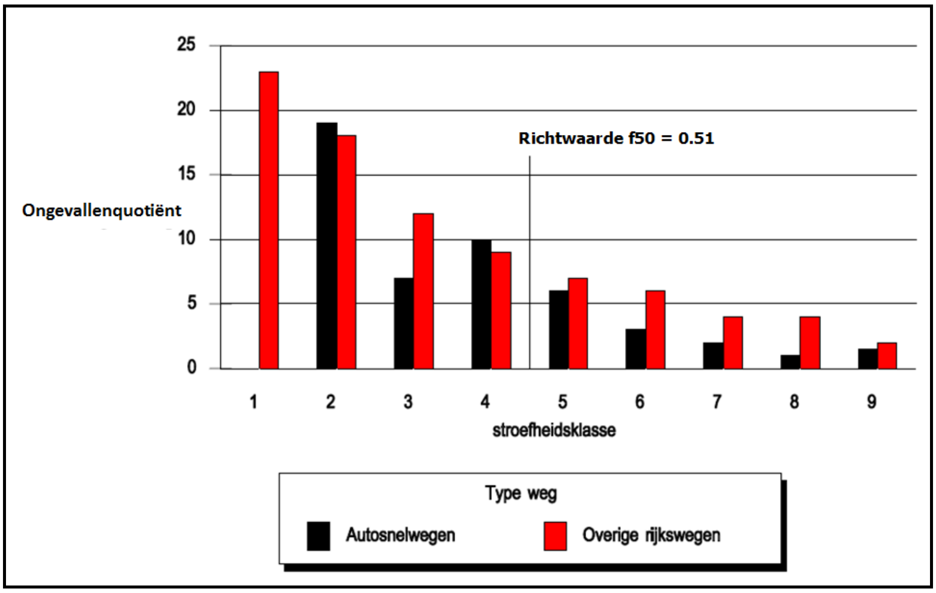


Figure 3-22: Road accidents versus skid resistance classes on main motorways and other national roads.

Table 3-6: Skid resistance values associated with the nine classes of skid resistance.

Class	Skid resistance value
1	< 0.36
2	0.36 - 0.41
3	0.41 - 0.46
4	0.46 - 0.51
5	0.51 - 0.55
6	0.56 - 0.61
7	0.61 - 0.66
8	0.66 - 0.71
9	> 0.71

According to a British study performed by TRL (Transport Research Laboratory) between the skid resistance and the crash rate on dry and wet surface conditions, the following figure 3-23 was extracted. Care must be taken in using the data together with the previously presented friction data. The friction coefficient SCRIM on the horizontal axis is not the same as the friction coefficient measured by the Dutch method earlier discussed in subsection 3.1.1. SCRIM stands for Sideway force Coefficient Routine Investigation Machine. SCRIM is a device that is mainly used in the UK to continually measure the skid resistance of the major road network. From figure 3-23 it can be seen that the level of skid resistance on dry road surfaces has little effect on the number of road accidents. The difference between the crash rate corresponding to a skid resistance value of 0.35 and 0.85 (extremes in figure 3-23) is very small (flat line). At the other side it can be seen that the level of skid resistance on wet road surfaces has a more distinct effect on the crash rate. The difference between the crash rate corresponding to a skid resistance value of 0.35 and 0.85 is on wet road surfaces much greater than on dry road surfaces. For higher levels of skid resistance (> 0.45 SCRIM) the crash rate on dry road surfaces is higher. For lower levels of skid resistance (< 0.45 SCRIM) the crash rate on wet road surface is higher. For further use of the data and relationship for the purpose of this study, the SCRIM data need to be converted to data according to the Dutch measuring method (86% retarded wheel performed at 50 or 70 km/h).

If the figures 3-22 and 3-23 are compared to each other, one can see that the crash rate - skid resistance sensitivity in figure 3-22 is much greater than the one depicted in figure 3-23 (the expression for the crash rate was identical). In figure 3-22 the crash rates vary in the range of 1.0 and 23.0 per mio vhc-km. The ones from figure 3-23 are in the range of 0.05 and 0.65 per mio vhc-km. A convincing explanation for this difference could not be found but it may be due to the fact that in this case ‘accident’ is defined in a different way (for example only fatal accidents). The only conclusion that was drawn, was that the pattern of figure 3-23 correlates to a safer road network than that of figure 3-22.

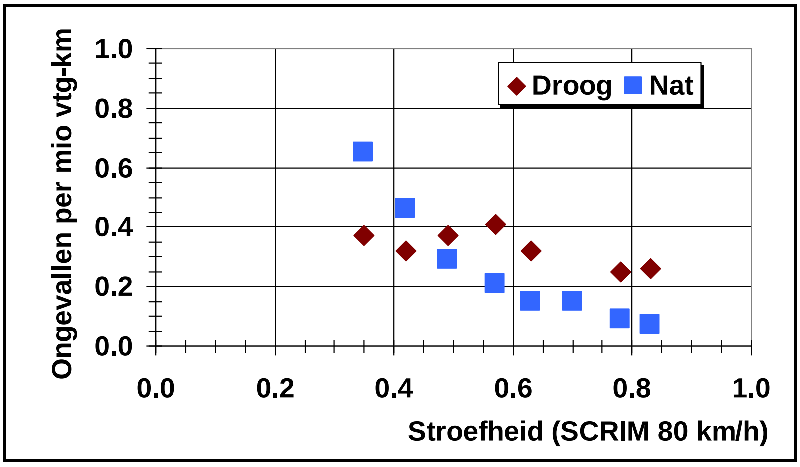


Figure 3-23: The correlation between the risk of accident and skid resistance in dry/wet road surface conditions.

3.3.2 Rut Depth vs. Crash Rate

In a study performed by CROW [Verkeersveiligheidsaspecten van wegoppervlakeigenschappen, 2005] on the relationship between the rut depth and the traffic accidents on motorways it was concluded that a rut depth of more than 17 mm on a wet road surface is significantly less safe than a rut depth of 16 mm or lower. On dry road surfaces no difference was found. Remarkable was that a rut depth in the range between 9 mm and 15 mm appeared to be much safer than a condition with a rut depth between 0 mm and 8 mm or larger than 16 mm [CROW, 2005]. This pattern is shown in figure 3-24. A possible explanation for this phenomenon can be a safer driving behaviour and a higher attention level in the rut depth range of 9 to 15 mm. This means that the anticipation of road users on observable road surface properties have once again a positive effect on the number of traffic accidents.

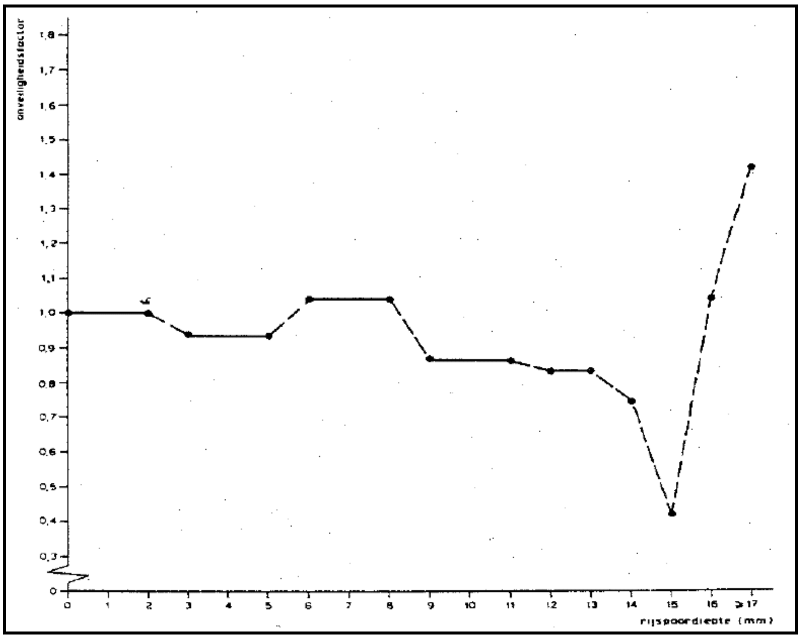


Figure 3-24: Correlation between the risk of accident and depth of ruts in mm (CROW, 2005).

In contradiction to this study, American studies revealed that the traffic safety improves with the increase of the rut depth. In a study [Road user effect model – influence of rut depth on traffic safety, 2011] performed by the Swedish National Road and Transport Research Institute regarding the relationship between road surface conditions and the crash rate it was found that the crash rate decreases with increasing rut depth with no increase in risk associated with rutting until deep ruts of more than 18 mm are reached. These findings are illustrated in figure 3-25. This figure also indicates that the effect of rut depth remains the same as traffic intensity increases. The higher the traffic class the smaller the slope of the curve. This Swedish relationship will be used in the model development presented in this report.

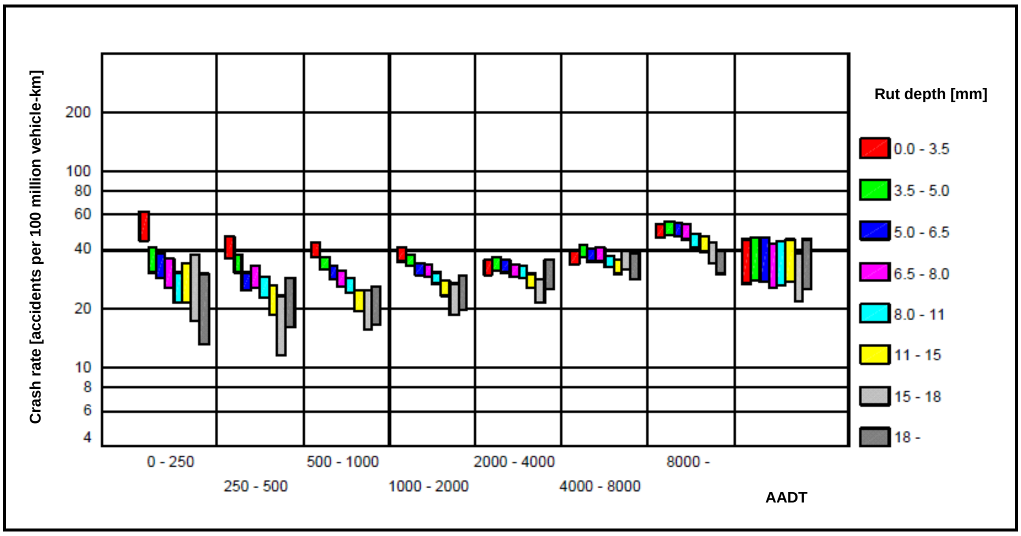


Figure 3-25: The expected crash rate in different rut depth and traffic classes.

Another study (Hollo and Kajtar, 2000) shows that if the rut depth increases, the risk of accident decreases. The correlation between the risk of accidents and depth of ruts [in mm] in dry and wet road surface conditions is shown in figures 3-26 and 3-27. A possible explanation for this experience is that deeper ruts are more visible for the road users which make them reducing their speed. This has as a consequence that the risk of accidents will decrease with developing rutting.

NB: The numbers for the expression of the crash rate in figure 3-23 to 3-27 are in the same range.

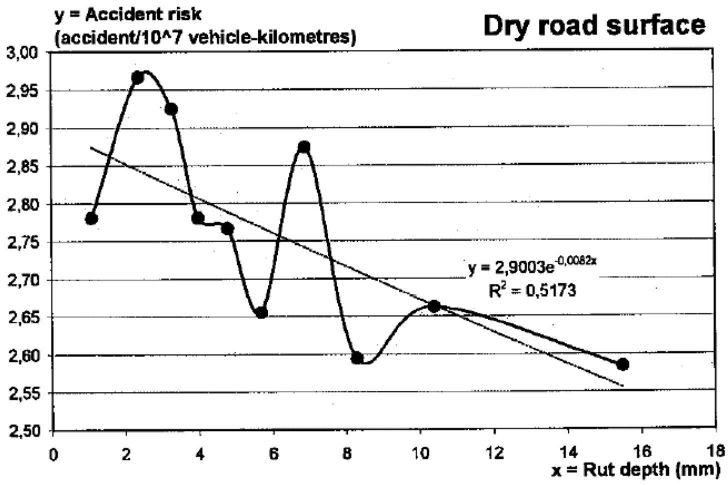


Figure 3-26: Correlation between the risk of accident and depth of ruts in mm in dry road surface conditions.

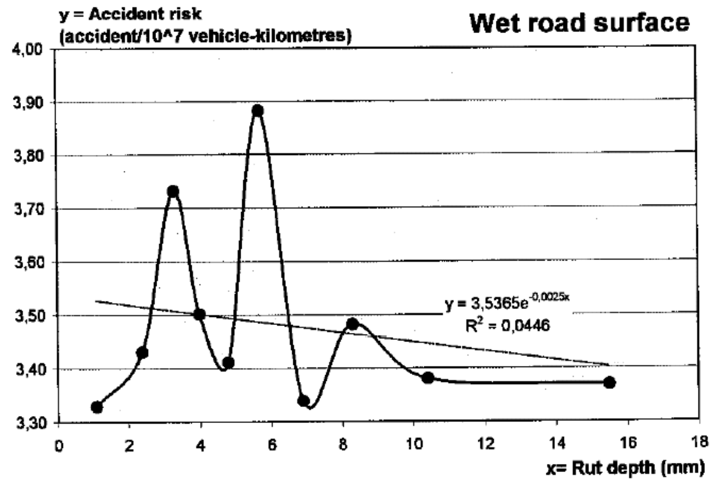


Figure 3-27: Correlation between the risk of accident and depth of ruts in mm in wet road surface conditions.

3.3.3 Roughness vs. Crash Rate

For analysis of the relationship between unevenness of pavement structures and the crash rate the same previously mentioned study [Road user effect model – influence of rut depth on traffic safety, 2011] was used. From figure 3-28 it can be deducted that the crash rate increases with increasing International Roughness Index (IRI). The figure also indicates that the rate of unevenness increases with increasing traffic intensity. The higher the traffic class the greater the slope of the curve. This Swedish relationship is used in the development of the model.

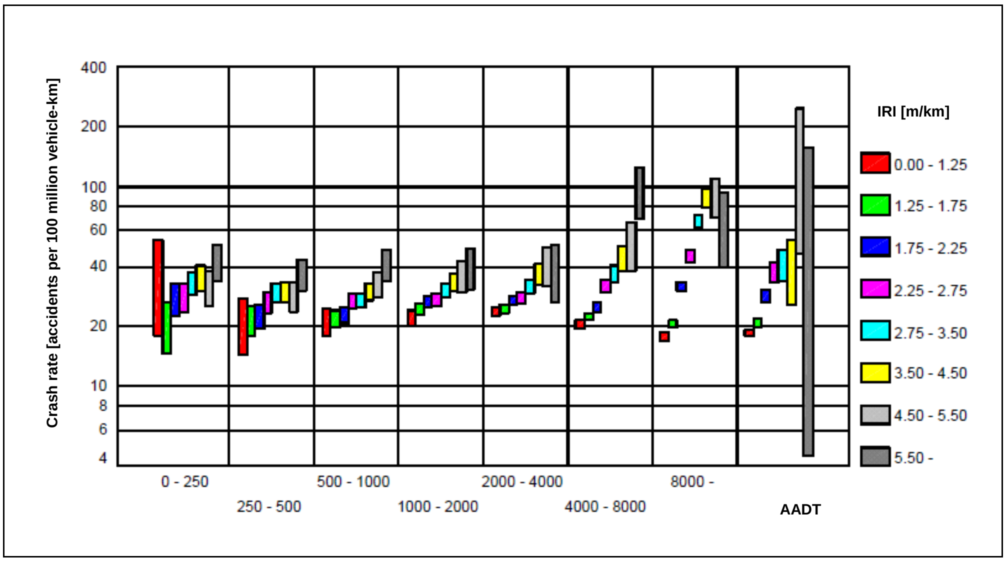


Figure 3-28: The expected crash rate in different IRI and traffic classes.

Another relationship between crash rate and roughness [Peter Cairney and Paul Bennett, ARRB Group] is shown in figure 3-29. This figure shows that a small increase in crash rate can be observed with increasing roughness till 120 counts/km. Beyond 120 counts/km the crash rate rapidly increases. The increase is more steep for roughness levels above 200 counts/km. The relationship can best be described with a polynomial function. The equation describing the polynomial function is:

$$\textbf{Crash rate} = 0.0049 \cdot X^2 - 0.4948 \cdot X + 33.468 \qquad \textit{(equation 5)}$$

Where:
X = roughness [counts/km]
Crash rate = number of accidents per 100 million vehicle-km

The expression for the roughness in counts/km (NAASRA) can be converted to IRI m/km with the following equation:

$$\textbf{IRI} = (\textbf{NAASRA counts} + 1.27)/26.49 \qquad \textit{(equation 6)}$$

Use of the conversion equation leads to the following predictive equation for the crash rate:

$$\textbf{Crash Rate} = 3.44 \cdot IRI^2 - 13.44 \cdot IRI + 34.1 \qquad \textit{(equation 7)}$$

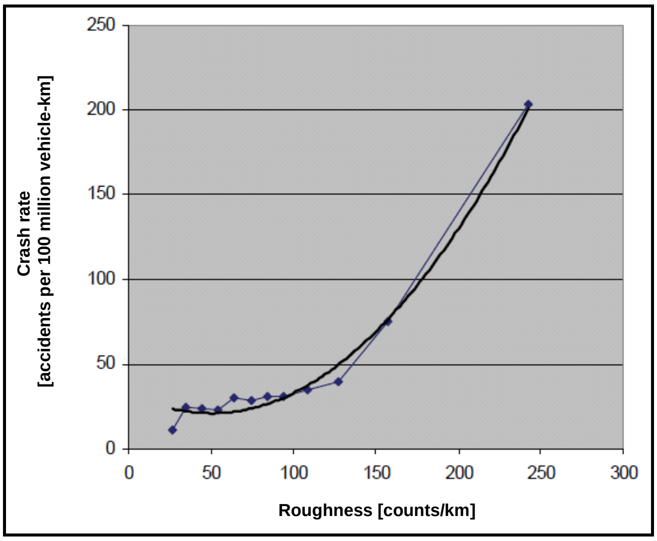


Figure 3-29: Relationship between roughness and crash rate [Peter Cairney and Paul Bennett, ARRB].

3.3.4 Texture Depth vs. Crash Rate

In a study conducted in Australia [Austroads, 2013] the relationship between texture depth (macro texture) and crash rate is studied. In this study it is stated that the crash risk is greater at sites with low macro texture. Besides this, it is also stated that the crash risk is relatively unaffected over a large range of the usual range of macro texture (0.5 – 50 mm). The results of the study shown that most of the road accidents happened within the texture depth range of 0.5 and 1.5 mm. Approximately 80% of the total number of road accidents happened within this range. This is the case for both wet and dry surface conditions as well as only wet condition in manoeuvre free areas e.g. straight mid block. The relationships between the texture depth and the crash rate for free areas can be seen in figure 3-30. The relationship between the texture depth and the crash rate for intersections and curves can be seen in figure 3-31. By comparing these two figures to each other, one can see that the correlated crash rate values for intersections and curves are greater. This indicates why the monitoring of skid resistance in curves is more important than in straight mid block.

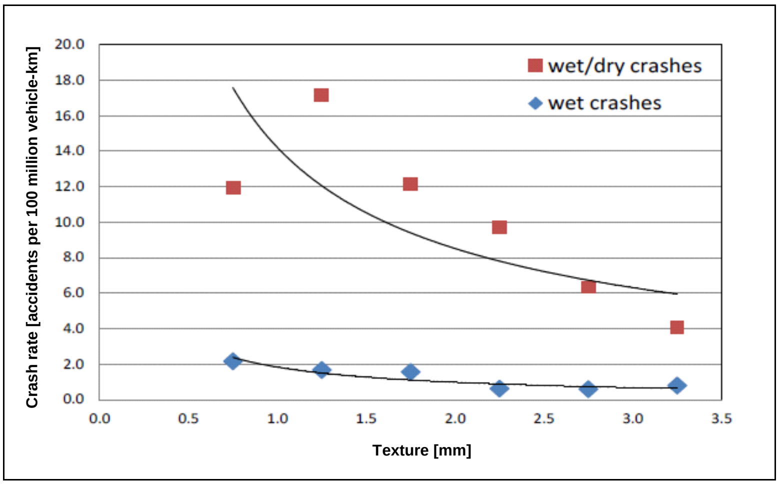


Figure 3-30: Crash rate vs. texture depth for manoeuvre free areas, both wet/dry and wet only crashes (Austroads, 2013).

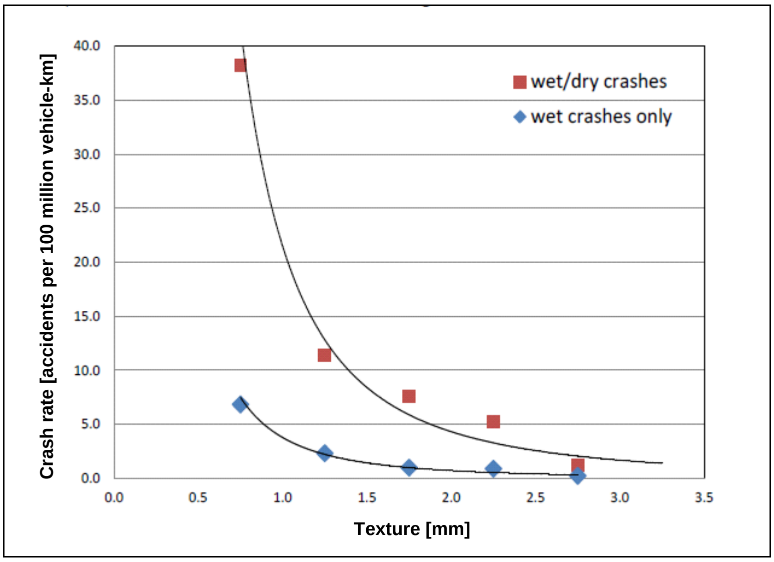


Figure 3-31: Crash rate vs. texture depth for intersection and curves, both wet/dry and wet only crashes (Austroads, 2013).

The correlation between texture depth and crash rate can be described with a power function. The functions for the crash rate for manoeuvre free areas and intersections & curves with wet/dry crashes are as following:

- Free areas with wet/dry crashes: Crash rate = $14.21 \cdot TD^{-0.74}$ *(equation 8)*
- Intersection and curve with wet/dry crashes: Crash rate = $21.49 \cdot TD^{-2.31}$ *(equation 9)*

These two equations will be used to develop the final model.

Figure 3-30 clearly demonstrates that the crash rate 'wet/dry' is much higher than the crash rate 'wet'. This can be explained as follows: the wet/dry crashes indicate both wet and dry crashes and the wet crashes indicates only the crashes that occurred when the road surface is wet. The wet crashes are presented alone in the figure just to illustrate the difference between wet/dry crashes and only wet crashes. The graphs also show that the number of wet crashes is also lower than that of the dry crashes. This is because the percentage of the time when the road surface is wet, is relatively shorter than the percentage of the time when the road surface is dry. The percentage of the time with a dry or wet road surface is illustrated in figure 3.32 (dry 5/6 & wet 1/6). The same figure also shows the percentage of the time during a year in which it rains or does not rain. These values correspond to the Dutch situation.

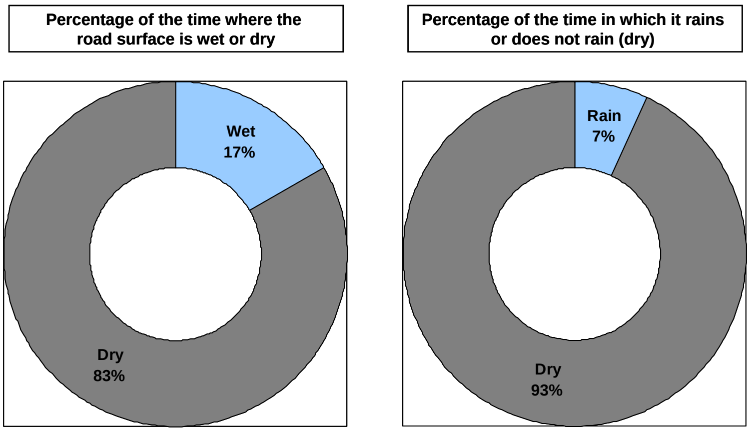


Figure 3-32: Percentage of the time where the road surface is wet or dry (left) and percentage of time during a year in which it rains or does not rain (right).

3.3.5 Water Film Depth Influence on the Crash Rate

Since water film thickness has a negative effect on the risk for traffic accidents, a model for predicting water film depth was queried. In this study the Gallaway model will be used for computation of the water film depth. [CROW 1997]. This model is developed for the computation of the water film depth across the carriageway during a rain shower. The data that need to be entered in this model are given here below.

- S = Crossfall / Slope [%]
- RI = Rainfall intensity [mm/hr]
- ETD = Estimated Texture Depth [mm]
- L = Width carriageway [m]
- WFD = Water film depth [mm]

The ETD can be calculated from the MPD by using the following equation:

$$\mathbf{ETD = 0.2 + 0.8\ MPD} \qquad \textit{(equation 10)}$$

The design value for the rainfall intensity in the Netherlands: 100 mm/hr (downpour). The general Gallaway equation is [Welleman, SCW publikatie L, 1977]:

$$\mathbf{WFD = 1.487 \cdot 10^{-2} * ETD^{0.11} * L^{0.43} * RI^{0.59} * (1/S)^{0.42} - ETD} \qquad \textit{(equation 11)}$$

The water film depth is calculated for three different crossfalls and eight different texture depths. The calculated water film depths can be seen in table 3-7.

Table 3-7: Calculated water film depth with texture depth and crossfall as variables (Gallaway model).

carriageway width 12 m (3 traffic lanes)		**Texture depth**							
		0.4 mm	0.6 mm	0.8 mm	1.0 mm	1.2 mm	1.4 mm	1.6 mm	1.8 mm
Crossfall	1.50%	3.06	3.01	2.93	2.82	2.70	2.57	2.43	2.28
	2.00%	2.66	2.60	2.51	2.39	2.26	2.12	1.97	1.81
	2.50%	2.39	2.36	2.21	2.08	1.95	1.80	1.65	1.49

3.4 Relationships Between Road Surface Properties and Fuel Consumption

In this section the effect of the road surface properties on the rolling resistance and fuel consumption by traffic will be addressed.

3.4.1 Texture Depth vs. Fuel Consumption

As was discussed earlier in subsection *3.2.2 'Fuel consumption'*, the texture depth is an indicative parameter for the fuel consumption. A literature study was conducted to see whether a reasonable relationship between texture depth and fuel consumption could be found. The relationships found are as following. In a study [Haaster, 2011] it is stated that an increase of 0.2 mm MPD leads to an increase of 6% of the rolling resistance and an increase of 1 unit in RMS leads to an increase of 7% of the rolling resistance. That same study clearly indicates that the texture parameters MPD and RMS are the relevant indicators for the rolling resistance. That study also states that the difference in rolling resistance coefficient between a texture depth of 0.3 mm and 2.0 mm amounts approximately 15%.
A study conducted by VTI [VTI, 2012] on the effects of the MPD-value on the fuel consumption presents that an increase of one unit MPD will increase the fuel consumption by 2.8% for cars and 3.4% for trucks.
Finally, according to a study [Benbow, E. et al, 2007] an increase of MTD (Mean Texture Depth) by 0.44 mm increases the rolling resistance by 3.6%. This 3.6% increase results in an increase of 0.72% fuel consumption.

Figure 3-33 [Hans Bendtsen, 2000] shows a relationship between the texture depth and the rolling resistance coefficient. From this figure it can be concluded that as the texture depth increases the rolling resistance coefficient will increase as well, as was to be expected. The relationship presented in figure 3-33 is based on measurements with an average weight passenger car. A heavier passenger car will of course attain a higher rolling resistant coefficient than a light passenger car. In this study the MPD (texture depth) will be used as the principal indicator for the calculation of the fuel consumption. The following steps that must be taken to predict the fuel consumption on SMA, DAC and PA roads are listed below:

- determine the progression of the texture depth with time of the three types of wearing courses.
- determine the rolling resistance coefficient which corresponds to the three types of wearing course.
- and finally determine the sensitivity of the fuel consumption to the rolling resistance coefficient found in step two by making use of figure 3-34.

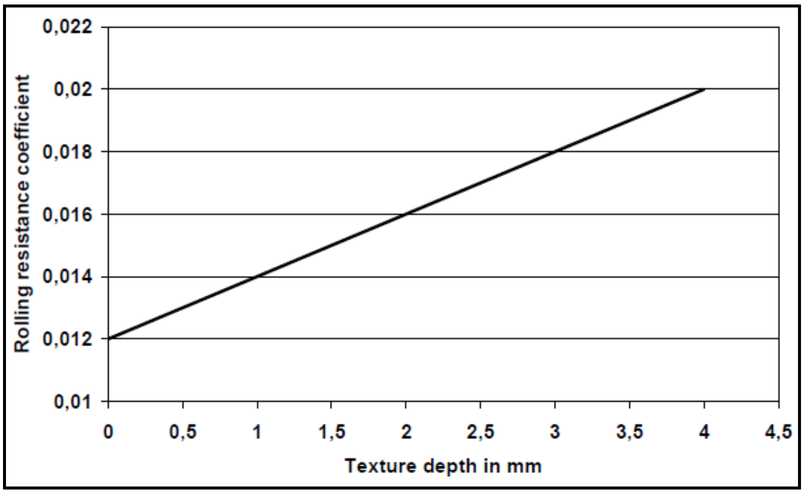


Figure 3-33: Relationship between texture depth and rolling resistance coefficient (H. Bendtsen, 2000).

However, it must be remarked that a small-size vehicle does not burn an equal amount of fuel as a large-size vehicle. Furthermore, the fuel consumption of a vehicle depends on how fuel efficient (economical) the vehicle itself is.

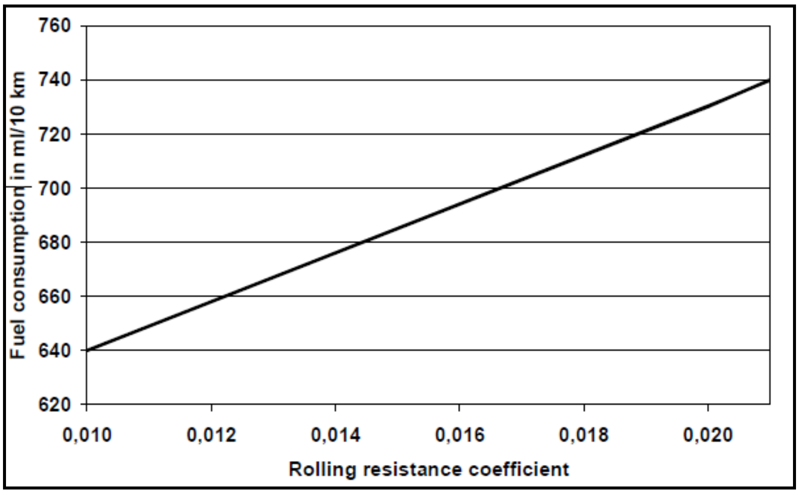


Figure 3-34: Relationship between rolling resistance coefficient and fuel consumption (H. Bendtsen, 2000).

The relationship between the rolling resistance and texture depth (figure 3-33) can be described with the following equation:

$$\mathbf{RRC = 0.002 \cdot Texture\ depth + 0.012} \qquad \textit{(equation 12)}$$

The relationship between fuel consumption and the rolling resistance (figure 3-34) can be described with the following equation:

$$\mathbf{Fuel\ consumption = 90.909 * RRC + 5.491} \qquad \textit{(equation 13)}$$

The equation for the fuel consumption can be simplified and becomes:

$$\mathbf{FC = 0.1818 * TD + 6.582} \qquad \textit{(equation 14)}$$

Where:
FC = fuel consumption [l/100 km]
TD = texture depth [mm]

3.4.2 Roughness vs. Fuel Consumption

The effect of roughness on the fuel consumption will be described in this part with values found in a literature study. The earlier mentioned study [Haaster, 2011] describes in subsection 3.4.1 that an increase of IRI of 1.0 m/km leads to an increase of 1.2% fuel consumption. In an article [Groenendijk, 2012] it is stated that an IRI drop of 1.0 m/km (from 2.1 to 1.1) leads to a decrease of 2.5% fuel consumption. In a study performed by VTI [VTI, 2012] it is shown that if the IRI value grows with one unit, the fuel consumption will increase at a rate of 0.8% for cars and 1.7% for trucks.

Figure 3-35 [Foley and Mclean,1998] shows the relationship between roughness and fuel consumption by a model that has been developed by ARRB Transport Research. This figure shows a small increase in fuel consumption between the IRI range 1.5 – 3.5 m/km followed by a decline from an IRI value of 3.5 or more. This decline is probably caused by slower driving speeds at rough roads. The relationship found in this study for the IRI range 1.5 – 3.5 m/km is copied into this report to calculate the fuel consumption due to the roughness . The reason for using this approach is that the limit value for the roughness in the Netherlands is 3.5 m/km. By plotting a trend line (orange dotted line) it can be seen that the corresponding level of fuel consumption lays between 11.85 litres/100 km for an IRI value of zero and 12.10 litres/100 km for an IRI value of 3.5. Due to the fact that these values are way too high in comparison with the average fuel consumption for passenger cars nowadays, it is decided to use a correction factor for these values. In order to determine the correction factor, figure 3-36 have been used (Stichting Bovag-Rai Mobiliteit). This figure presents the average value fuel consumption of the top 50 best selling petrol cars in the Netherlands. From this figure, the average value fuel consumption has been calculated for the last 10 years (from 2000 to 2010). This results in an average value of 6.5 litres/100 km. By using this value the correction factor can hereafter be determined. This results in a correction factor of 0.54. So the values 11.85 l/100 km and 12.10 l/100 km mentioned earlier become 6.4 l/100 km and 6.53 l/100 km, respectively. This means an increase of approximately 0.6% fuel consumption per 1.0 IRI increase. This value is almost equivalent to the value found in the VTI study.

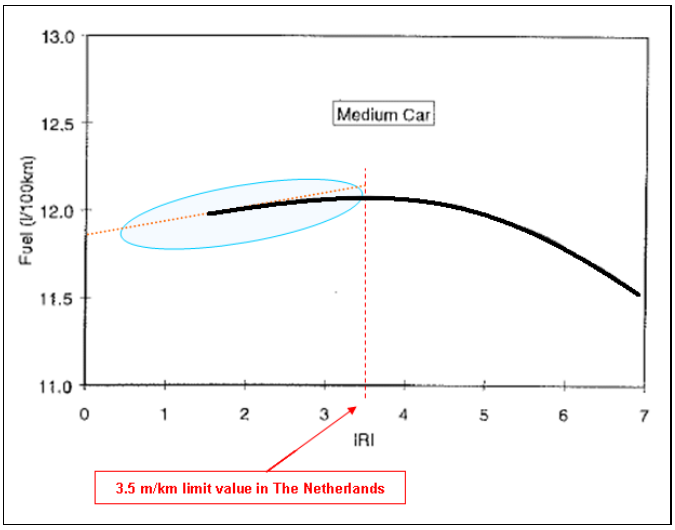


Figure 3-35: Relationship between roughness and fuel consumption (Foley and Mclean,1998).

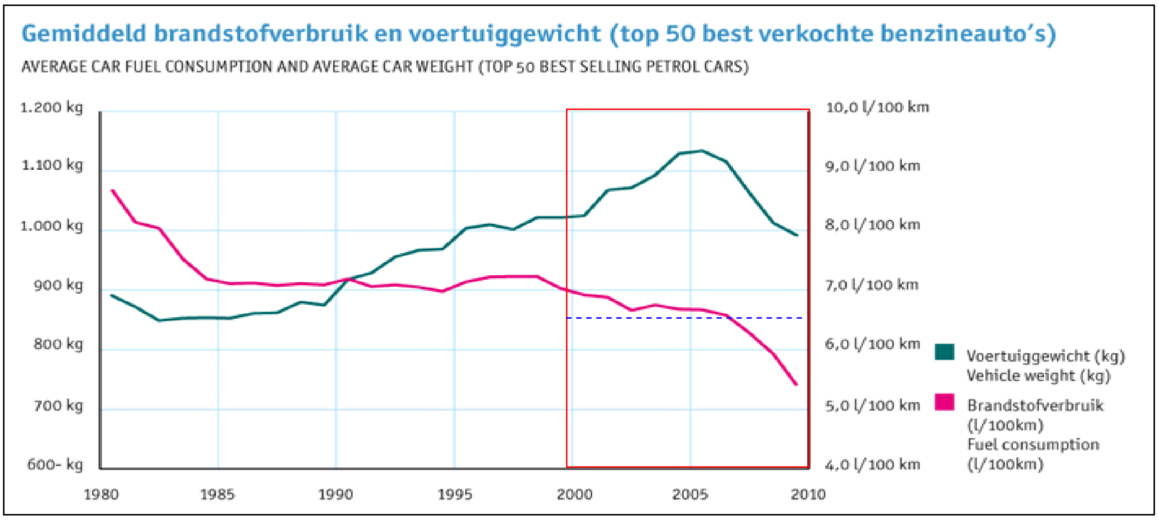


Figure 3-36: Average car fuel consumption and average car weight (Source: Stichting Bovag-Rai Mobiliteit).

For the calculation of the fuel consumption in the final model it is decided to use the relationship describing the plotted orange dotted line shown in figure 3-35 combined with the correction factor. The equation describing this line is the following:

$$\mathbf{FC = 0.0386 * IRI + 6.40} \qquad \textit{(equation 15)}$$

<u>Where:</u>
FC = fuel consumption [l/100 km]
IRI = International Roughness Index [m/km]

3.5 Relationships Between Road Surface Properties and Traffic Noise

In this part of the report the relationships between the road texture depth and the traffic noise found in the literature study is discussed. At the end, the relationship will be described with an equation which will be added to the final model.

3.5.1 Texture Depth vs. Traffic Noise

According to an Australian study [Wayson, 1998] between the texture depth and the traffic noise, the following figure 3-37 is extracted. The information in this figure is acquired using the Statistical Pass-by Method (SPM). SPB is a method to accurately determine the acoustic properties of a road surface for passenger cars and trucks. From figure 3-37 it can be seen that the texture depth is a key variable in noise generation. This is the case both for bituminous and concrete surfaces. Remarkable is the different slope or relationship when comparing texture depth and noise levels for concrete pavements compared to asphalt pavements. This study will only focus on asphalt pavements.

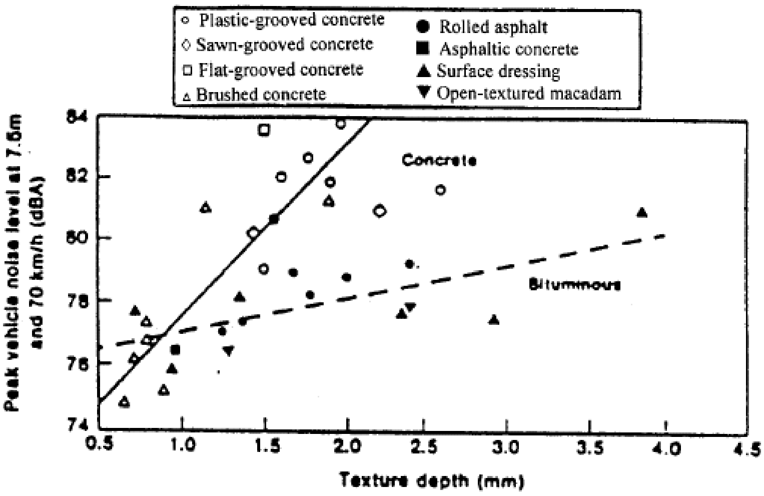


Figure 3-37: Relationships between texture depth and noise for various bituminous and concrete surfaces.

The slope or the relationship between the texture depth and the noise level for the asphaltic pavements can be described with the following equation:

Noise level [dBA] = 1.143 · TD [mm] + 75.93 *(equation 16)*

This equation will be used in the final model to determine the noise level that can be expected from asphalt pavements. This equation will be used for dense asphalt wearing courses (DAC & SMA). For porous asphalt wearing courses this equation will also be used but 3 dB(A) will be deducted from the calculated noise level. Due to the fact that pavement texture does not remain the same over time (clogging, raveling, polishing), the effect of the texture change on the tire-pavement noise levels must be evaluated during the service life of asphalt pavements.

The acoustic performance of wearing courses can also be determined by using the two Standard Calculation Methods (SRM 1 and 2) described in Publication 316 of CROW [CROW, 2012]. This publication describes the effect of wearing courses on the produced traffic noise due to passing vehicles. The effect of the wearing course on the traffic noise is indicated by a correction factor, the so called road surface correction factor. A road surface is called silent when the produced traffic noise on that corresponding surface is less than that of a standard road surface of dense asphalt (reference wearing course) with the same traffic intensity and speed. The difference in noise production is called the road surface correction factor. A wearing course that produces less noise than the reference wearing course has a negative road surface correction factor. The reference wearing course include all DAC gradings and SMA from a grain size of 11 mm or larger. In this study, the standard calculation method one (SRM-1) will be used. The equations that correspond to this method in order to determine the road surface correction factor are the following:

$$\mathbf{C_{road\ surface,m}\ (v_m) = C_{initial,m}\ (v_m) + C_{time,m}} \quad \textit{(equation 17)}$$

$$\mathbf{C_{road\ surface,m}\ (v_m) = \sigma_m + \tau_m \log (v_m / v_{0,m})} \quad \textit{(equation 18)}$$

$$\mathbf{C_{initial,m}\ (v_m) = \Delta L_m + \tau_m \log (v_m / v_{0,m})} \quad \textit{(equation 19)}$$

Where:
$C_{road\ surface,m}$ = road surface correction [dB(A)]
$C_{initial,m}$ = initial correction factor [dB(A)]
$C_{time,m}$ = ageing correction [dB(A)]
σ_m = parameter [-]
τ_m = parameter [-]
v_m = speed of traffic [km/h]
$v_{0,m}$ = reference speed [km/h]
ΔL_m = parameter [-]

In order to determine what is the average increase of noise level per year for porous asphalt layers, the ageing correction $C_{time,m}$ has to be calculated. The $C_{time,m}$ can be calculated by subtracting the initial correction from the road surface correction ($C_{road\ surface,m}$ - $C_{initial,m}$). However it must be said that the $C_{time,m}$ indicates the increase of noise level during 75% of the average service life of porous asphalt wearing courses. In order to calculate the increase of noise level per year, the obtained $C_{time,m}$ should be divided by 8.25 years. This is due the fact that PA wearing courses have an average service life of 11 years (according Rijkswaterstaat).

In the first standard calculation as well as in the second standard calculation, a distinction is made between three vehicle categories. Each vehicle category has its own specific reference speed. The three vehicle categories and their corresponding reference speed are given below.

Vehicle category:	light (m = 1)	middle (m = 2)	heavy (m = 3)
Reference speed:	80 km/h	70 km/h	70 km/h

The values of the parameters for the equations 17, 18 and 19 are given in the tables A-1, A-2 and A-3 (see Appendix A). In order to see what is the ageing correction factor for a porous asphalt layer the following calculation is given. The calculation consists of three steps.

- **1st step: Calculation of $C_{road\ surface,m}$ for porous asphalt layer:**

$C_{road\ surface,1}$ = -1.4 – 6.5 log(v_1/80) v_{min}=50 km/h & v_{max} = 130 km/h
$C_{road\ surface,2,3}$ = -3.1 + 0.2 log($v_{2,3}$/70) v_{min}=70 km/h & v_{max} = 100 km/h

Table 3-8: $C_{road\ surface,m}$ (v_m) for porous asphalt layer in dB(A).

	30 km/h	40 km/h	50 km/h	60 km/h	70 km/h	80 km/h	90 km/h	100 km/h	110 km/h	120 km/h	130 km/h
m=1	-	-	-0.1	-0.6	-1.0	-1.4	-1.7	-2.0	-2.3	-2.5	-2.8
m=2,3	-	-	-	-	-3.1	-3.1	-3.1	-3.1	-	-	-

- **2nd step: Calculation of $C_{initial,m}$ for porous asphalt layer:**

$C_{initial,1}$ = -3.0 - 6.5 log (v_1 /80)
$C_{initial,2,3}$ = -4.5 + 0.2 log ($v_{2,3}$ /70)

Table 3-9: $C_{initial,m}$ (v_m) for porous asphalt layer in dB(A).

	30 km/h	40 km/h	50 km/h	60 km/h	70 km/h	80 km/h	90 km/h	100 km/h	110 km/h	120 km/h	130 km/h
m=1	-	-	-1.7	-2.2	-2.6	-3.0	-3.3	-3.6	-3.9	-4.1	-4.4
m=2,3	-	-	-	-	-4.5	-4.5	-4.5	-4.5	-	-	-

- **3rd step: Calculation of $C_{time,m}$ for porous asphalt layer:**

$C_{time,m} = C_{road\ surface,m}(v_m) - C_{initial,m}(v_m)$

Table 3-10: $C_{time,m}$ (v_m) for porous asphalt layer in dB(A).

	30 km/h	40 km/h	50 km/h	60 km/h	70 km/h	80 km/h	90 km/h	100 km/h	110 km/h	120 km/h	130 km/h
m=1	-	-	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6	1.6
m=2,3	-	-	-	-	1.4	1.4	1.4	1.4	-	-	-

From the values given in table 3-8 it can be seen that porous asphalt layers produce indeed a benefit in noise reduction (all the values are negative). For the maximum speed of the three vehicle categories, the road surface correction factor corresponds to a benefit of approximately 3 dB(A) noise reduction (-2.8 dB(A) and -3.1 dB(A)).
Table 3-10 shows that the ageing correction for the light vehicle category corresponds to 1.6 dB(A) and 1.4 dB(A) respectively for the middle and heavy vehicle category. This means that the increase of noise level per year for porous asphalt is 0.19 dB(A) for category m=1 and 0.17 dB(A) for the categories m=2 and m=3.

Instead of using equation 16 to determine the noise level that can be expected from porous asphalt pavements, the average value of these values (0.19 dB & 0.17 dB) will now be used to formulate an equation that describes the increase of noise level during the service life of porous asphalt wearing courses. The equation is the following:

$$\textbf{Noise level [dBA]} = dB_{initial} + 0.18 * t \qquad \textit{(equation 20)}$$

Where:
$dB_{initial}$ = initial noise level (t=0)
t = age of wearing course [year]

4. DATA SELECTION AND PROCESSING

This chapter describes the selection and the construction of the database that will be used in this study for the development of the final model. The first section of this chapter lists some requirements to be set to the database enabling setting up an ideal structure for meeting the objectives of the study. In the sections 4.2 to 4.5 the used database will be described. The data used from the selected database will also be discussed.

4.1 Database Selection

This section will address the requirements for the database selection. This analysis will lead to a list of available databases to be chosen from for the purpose of the study. For each available database advantages and disadvantages will be mentioned. In the last part of the section the most appropriate database will be selected for this study.

4.1.1 Requirements for Database Selection

The main objective of the study is to develop a model that can be used as an indicative tool of what pavement engineers, contractors and roadway authorities may expect from the designed asphalt pavements in terms of traffic safety, fuel consumption and service life. For the development of the model it is of major importance to have available periodic test data of pavement structures and road surface properties preferably with one year intervals. These data provide information on the rate of change of pavement and road surface properties with time or traffic. The more data of consecutive years are available, the more accurate the decline of the road surface properties can be mapped. Other requirements for the database selection are the following:

- Type of wearing course must be known;
- Type of mineral aggregates used in wearing course must be known;
- Availability of test data for major roads and provincial roads;
- The traffic intensity on the evaluated asphalt pavements must be known;
- Year of construction of wearing course.

The presence of traffic data in the database is very important because the literature review showed that the development of some road surface properties is dependent on the traffic intensity. Besides the traffic intensity, also other data should be available to have an idea which parameters are the best predictor of the development of the road surface properties. These data are:

- Asphalt layer thickness;
- Type of subgrade;
- Traffic growth rate.

Care must be taken in the use of data from abroad. These data must at least be collected under conditions similar to the Dutch situation and the pavement structures and mineral aggregates used should also have some similarity. To avoid potential discrepancies it is desirable to evaluate only Dutch road sections or at least as much as possible.

4.1.2 Databases

Databases that contain test data (road surface properties) of consecutive years are not so easy to obtain. Most of the time these databases are confidential and managed by companies that perform pavement testing on a regular basis. Because KOAC•NPC is one of the companies participating in the research project, plenty of data became available. So one of the databases that will be used for this study is the KOAC•NPC database. This database contains data on skid resistance, texture, longitudinal profile (roughness) and transverse unevenness (rutting).

Another database that can be used for the study is that of a Long-Term Pavement Performance Program (LTTP). A LTTP is a large research program that collects and investigates pavement related details that are critical for the pavement performance. A well known LTTP project performed in the Netherlands is the Strategic Highway Research Program – the Netherlands (SHRP-NL). The SHRP-NL database contains data on Falling Weight Deflection measurements (FWD), Automatic Road Analyser

(ARAN) measurements. For this study the FWD data are of no relevance. Only the ARAN measurement data are needed.

4.1.3 Advantages and Disadvantages of the Databases

The KOAC•NPC database contains a large number of test data performed on a large selection of roads in the Netherlands. The large selection of roads makes this database ideal to be used for this study. The disadvantage of the KOAC•NPC database is that information about the pavement structure itself is usually not available, because this type of information is not being shared by the clients to the contractor which is in this case KOAC•NPC, the organisation performing the measurements. This implies that in many cases the type of wearing course is not known, nor is the total thickness of the asphalt pavement structure. For the skid resistance it is important to know which mineral aggregate is used in the wearing course otherwise it is hardly possible to accurately predict the decline of skid resistance with time. Another disadvantage is that most of the measurements are performed for only a single moment and not for consecutive years.

One of the advantages of the SHRP-NL database is that this database consists of more than 250 road sections. This road sections are situated on motorways, provincial and rural roads. Multiple FWD and ARAN measurements were performed on these road sections each year within the period 1990-2000. Data about the pavement structures, materials, climatic conditions and traffic volume can all be found in the SHRP-NL database. This makes the SHRP-NL database very appropriate to be used in this study.

4.1.4 Chosen Databases for This Study

For this study the decision was taken to use both the KOAC•NPC database and the SHRP-NL database. The first database contains ample information on the development of skid resistance and texture. For the longitudinal profiles (roughness) and the transverse profiles (rutting) both databases will be used. These two databases contain road sections with a variety of wearing courses and subgrades making them very useful.

4.2 KOAC-NPC Database

Each year, KOAC-NPC laboratories and pavement testing group perform various measurements and tests on road pavements. These measurements are performed to test all the characteristics that may apply to the design and construction of roads. The measurements are performed to capture pavement surface data and to record the structural condition of pavements. KOAC-NPC is approached by different clients for monitoring and investigating the functional pavement surface properties. Later, on the basis of the results obtained from the measurement data, advice is provided to the clients.

Some of the performed measurements on pavement surface properties are the following: skid resistance, texture, roughness, rutting, ravelling, traffic noise and markings. All these measurement data were put together in a database (KOAC-NPC database). By collecting tested data from different years (same project), this compiled information resulted in a database that also contains test data of consecutive years needed for development of performance curves. This database is confidential and is only managed by KOAC-NPC.

For this study it was decided to mainly extract data of skid resistance and texture (MPD) from the KOAC-NPC database. The necessary data came from projects where measurements were carried out in consecutive years. Measurements data of three years or more are suitable for this study By collecting such data, a better indication for the progression of the skid resistance or texture depth can be obtained.

4.3 SHRP-NL Database

The SHRP project was initiated in the USA in 1990 and joined by several other countries worldwide. The primary goal of the project was to improve the state of knowledge on pavement materials and pavement structures with as secondary goal to design and construct better roads and apply better suited maintenance measures. Besides these objectives, the SHRP project had the goal to increase knowledge about the long-term performance of pavements. The American SHRP project is called the SHRP-US in this report and the Dutch one is called SHRP-NL. The two programmes use the same basic structure but the American one is much larger than the Dutch one and focuses on more issues than the Dutch version did. The SHRP-NL programme paid quite some attention on studying the performance of roads on the long term and the effect of various types of maintenance measures.

The SHRP-NL database consists of more than 250 selected road sections across the Netherlands. On these road sections measurements were performed over a period of 10 years. This long period provides data on the change of distress with time and data on the condition of roads after maintenance was applied. The overall goal of the project was to develop an approach for more efficient spending of maintenance budgets.
The road sections in the Netherlands each have a standard length of 300 m and are situated on a straight section of the road with no curves or bends. Within the 300 m falling weight deflection testing was performed on fifteen stations with a 20 m station spacing. The first station was located 10 m after the zero of the section.
To acquire more information about the materials used and the thicknesses of the pavement layers, cores were taken from the road sections at the positions 25 m before the start of the section and 25 m after the end of the section.

In addition to FWD, also ARAN measurements and visual inspections were performed. The ARAN measurements were used to collect data on the longitudinal and transverse unevenness of the road. These measurements also aimed to verify if the distresses that can be seen on the video tape match with those of the visual inspection.
The FWD measurements are not relevant in the framework of this study. The visual inspections had as target to register observable damage, such as e.g. the extent and degree of cracking in the road section. The results of the visual inspection of each road section will be used later for verification whether residual lives of the road sections, calculated by the developed models match with the predicted extent of distress.

4.4 A70 Bamberg Database

On a road section A70 in Bamberg, Germany skid resistance measurements were performed to see how skid resistance would develop with time. The measurements were performed on a DAC and a SMA wearing course with different grading. The road section A70 has an annual average daily traffic intensity of 35000 vehicles. Further, the A70 has two traffic lanes in each direction (see figure 4-1). Over the total grading range of the stones the polishing sensitive mineral aggregate basalt is used (PSV 47). The measurements were performed in the period between September and November. Testing was limited to this short period to limit the seasonal variation. The results of the processed data obtained from the Bamberg database can be seen in Chapter 6.

Figure 4-1: A70 Bamberg Germany.

4.5 N732 Lonneker-Losser Database

Similar skid resistance measurements were performed on the provincial road N732 between Lonneker and Losser. The measurements were carried out in the right-hand wheel path. The N732 has an annual average daily traffic intensity of 6700 vehicles. The percentage of truck traffic on this road section is 7% and the rest is passenger car traffic. The N732 consists of two traffic lanes, one in each direction (see figure 4-2). The measurements were performed on DAC and PAC wearing courses with moraine as mineral aggregate and on an SMA wearing course with Greywacke as mineral aggregate. These measurements data will be processed and added to the final model. The results of the analysis can be seen in Chapter 6.

Figure 4-2: Provincial road N732 between Lonneker and Losser (above Enschede).

5. RESEARCH METHOD

In this chapter, the research method will be described and visualized with the help of a flowchart. At the end, the mini models that will be used in this study to develop the final model will be described. These mini models are the results of the processed data.

5.1 Visualization of Research Method

The research method starts with the processing of the data obtained from the used databases. This processing must be carried out to determine the development of the road surface properties with time and the performance of the mineral aggregates used in wearing courses. The development of the road surface property ‘crossfall’ is not evaluated because this property hardly changes with time (unless there are unequal settlements in the cross-section). The development of the four other road surface properties will be described with the use of equations (10 mini models). The performance of the tested mineral aggregates will also be described by means of equations (13 mini models). This results in a total of 23 mini models. The next section will elaborate on these mini models.

The 23 mini models, together with the seven relationships between the road surface properties and the environmental (sustainability) aspects found in the literature study are used to develop a draft model. The Gallaway model used to determine the water film thickness on asphalt pavements is also used to develop the draft model. Finally the draft model is tested to check whether it predicts proper results. If this is the case, it can be considered that the final model is obtained. The flowchart corresponding to the research method of this study can be seen in figure 5-1.

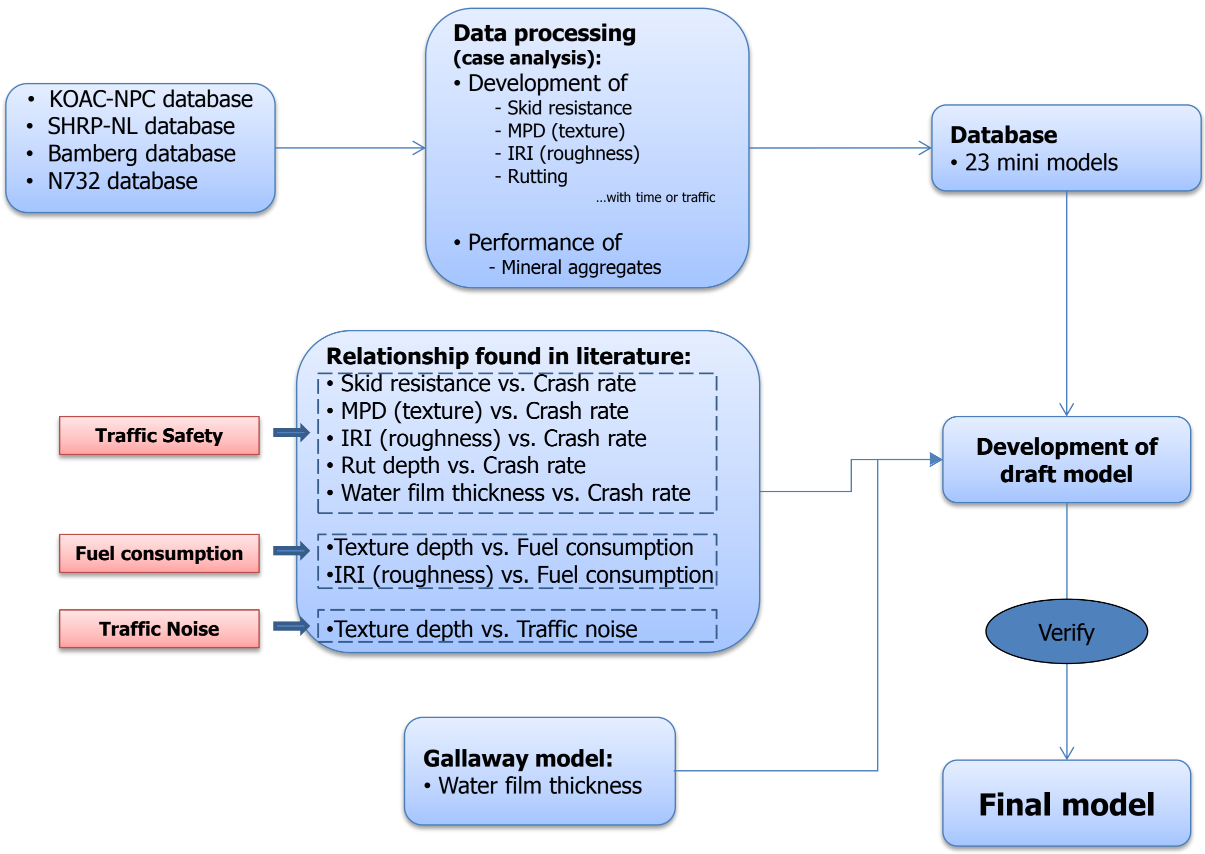


Figure 5-1: Flowchart of research method.

5.2 Mini Models

The final model that will be developed in this study consists of 23 mini models. The first 13 mini models describe the performance of mineral aggregates used in wearing courses. In this study the performance of eight mineral aggregates was evaluated (polishing rate). The polishing rate of the mineral aggregates indicates the development of the skid resistance. The eight evaluated mineral aggregates are given in table 5-1. This table shows that data of several mineral aggregates used in some of the wearing courses were not available. For this reason no mini model could be obtained. Two mini models for the mineral aggregates greywacke and moraine in SMA wearing course could be found. Beware, mineral aggregates with different PSV are evaluated (moraine 53 and moraine 44).

Table 5-1: Mini models: performance of mineral aggregates (indicate the skid resistance).

Type of mineral aggregate	**Wearing course**		
	PAC	**SMA**	**DAC**
Greywacke	Yes (mini model 1)	Yes (mini models 5+6)	No
Dutch crushed gravel	Yes (mini model 2)	No	No
Porphyry	Yes (mini model 3)	No	No
Moraine	Yes (mini model 4)	Yes (mini models 7+8)	Yes (mini model 12)
Bestone	No	No	No
Basalt	No	Yes (mini model 9)	Yes (mini model 13)
Diabase	No	Yes (mini model 10)	No
Dolomite	No	Yes (mini model 11)	No

The last 10 mini models describe the development (progression) of the road surface properties texture (MPD), roughness (IRI) and rutting. They are the following:

- The development of the texture for three types of wearing course (table 5-2)
- Development of roughness for two asphalt thicknesses (thin and thick) on soft (clay) and stiff (sand) subgrade (see table 5-3)
- Development of rut depth in three types of wearing course (see table 5-4)

Table 5-2: Mini models: development of texture (MPD).

Wearing course	**Texture (MPD)**
DAC	Mini model 14
SMA	Mini model 15
PAC	Mini model 16

Table 5-3: Mini models: development of roughness (IRI).

Thickness asphalt	**Subgrade**	
	Clay (soft)	**Sand (stiff)**
thin	Mini model 17	Mini model 19
thick	Mini model 18	Mini model 20

Table 5-4: Mini models: development of rutting.

Wearing course	**Lower layers**	**Mini-model**
DAC	DAC	21
SMA	DAC	22
PAC	DAC	23

5.3 Equations and Matrices Used in the Model to Describe the Relationships

The equations and matrices that will be used to describe the relationships between the road surface properties and the environmental aspects traffic safety (crash rate), fuel consumption and traffic noise are the following:

- **Skid resistance vs. Crash rate**

Crash rate [crashes/100 mio.veh.km] = $27.239 \cdot 10^3 \cdot e^{-7.55 * SR}$ Motorway *(equation 21)*
Crash rate [crashes/100 mio.veh.km] = $13.37 \cdot 10^3 \cdot e^{-5.46 * SR}$ Provincial roads *(equation 22)*

- **MPD (texture) vs. Crash rate**

Free areas with wet/dry crashes

Crash rate [crashes/100 mio.veh.km] = $14.21 \cdot TD^{-0.74}$ *(equation 23)*

Intersection & curve with wet/dry crashes

Crash rate [crashes/100 mio.veh.km] = $21.49 \cdot TD^{-2.31}$ *(equation 24)*

- **IRI (roughness) vs. Crash rate**

Table 5-5: Matrix: IRI vs. Crash rate.

AADT \ IRI	0 - 1.25 [m/km]	1.25 - 1.75 [m/km]	1.75 - 2.25 [m/km]	2.25 - 2.75 [m/km]	2.75 - 3.5 [m/km]	3.5 - 4.5 [m/km]	4.5 - 5.5 [m/km]	> 5.5 [m/km]
0 - 250	32	20	28	29	35	36	33	42
250 - 500	19	22	23	27	31	32	30	36
500 - 1000	21	22	23	27	28	32	34	40
1000 - 2000	22	24	27	28	32	34	36	38
2000 - 4000	24	24	28	29	33	37	39	37
4000 - 8000	21	22	26	33	38	44	51	93
8000 - 10000	18	20	33	45	67	88	88	60
> 10000	19	21	30	38	40	38	101	30

Table 5-5 presents the average values extracted from figure 3-28.

- **Rut depth vs. Crash rate**

Table 5-6: Matrix: Rut Depth vs. Crash rate.

AADT \ Rut depth	0 - 3.5 [mm]	3.5 - 5.0 [mm]	5.0 - 6.5 [mm]	6.5 - 8.0 [mm]	8.0 - 11.0 [mm]	11.0 – 15 [mm]	15- 18.0 [mm]	> 18.0 [mm]
0 - 250	53	35	34	32	27	28	26	20
250 - 500	41	36	28	30	28	23	17	22
500 - 1000	40	35	32	30	28	23	20	22
1000 - 2000	39	37	34	33	30	27	24	26
2000 - 4000	35	37	36	34	33	29	26	32
4000 - 8000	38	40	39	39	36	34	37	34
8000 - 10000	52	53	52	51	45	43	39	37
> 10000	37	38	38	36	37	38	32	37

Table 5-6 presents the average values extracted from figure 3-25.

- **Water film thickness Influence on the Crash rate**

Calculated water film depth + rut depth = new estimated rut depth

Basically the rut depth forms the input for the determination of the crash rate. However, when the Gallaway model predicts a water film thickness under critical rain conditions, the local situation will be more crash sensitive. For that reason the sum of the rut depth and the water film depth resulting from the Gallaway model will be used to determine the crash rate. This is done by using the matrix given in table 5-6. This approach is only used for non-porous asphalt concrete (DAC & SMA).

Earlier in subsection 3.3.2 it was discussed that the crash rate decreases with increasing rut depth. This means that by summing up the rut depth and the water film depth (results in a deeper rut), the crash rate will decrease and this in contrast with the statement that aquaplaning increases the risk of accidents. This also means that this method where the rut depth and the water film depth are summed up cannot be applied (contradiction). Besides this, the weight factor for the water film depth should be larger than one (>1.0) and in addition to this the Gallaway model does not include neither rut nor heaves alongside rut. Furthermore, a relationship indicating how does rut depth relate to the water film thickness could not be found. So in order to be able to determine the crash rate due to a water film layer on the road surface another method must be looked for.

Instead of using the rut depth as the indicative parameter to determine the crash rate due to a water film layer on a road surface, the skid resistance will now be used as the indicative parameter (new method). The idea behind this new method is to consider that a water film layer on a road surface leads to a lower friction coefficient (skid resistance). This referring lower friction coefficient is called in this model the 'reduced skid resistance'. With the help of this reduced skid resistance, the crash rate due to the water film layer can later be determined by using figure 3-22. The reduced skid resistance due to a water film layer on the road surface will be determined by using a correction factor. In order to find an equation with witch the correction factor can be determined, a relationship between the vehicle speed, water film depth and the skid resistance (friction coefficient) must be found.

Figure 5-2 [Sakai et al., 1978] shows a relationship between the vehicle speed, water film depths (0, 1, 2, 5 and 10 mm) and the 'locked braking force' coefficient (LBC). This figure appeared to be very helpful for the development of the required equation to determine the correction factor. The maximum vehicle speed tested by Sakai is 100 km/h. In the test, the effect of water film layers on the LBC is tested with two types of tires, a cross (diagonal) tire and a radial tire. In this study, the results corresponding to the radial tire will be used. The reason for using the results obtained from the radial tire is due to the fact that most vehicles have radial tires. From figure 5-2 it can be seen that the locked braking force coefficients decrease with increasing speed and increasing water film depth.

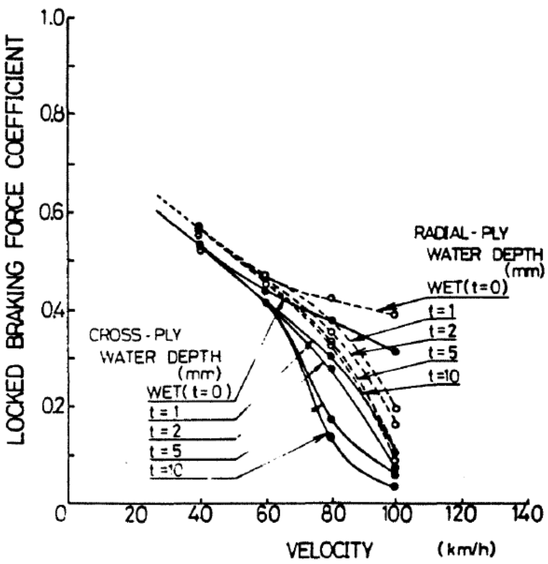


Figure 5-2: Relationships between vehicle speed, water film depths and locked braking force coefficient [Sakai et al., 1978].

For the development of the equation to determine the correction factor several steps must be followed. The first step is to create a table with the coefficients corresponding to the water film depths (0, 1, 2, 5 and 10 mm) and vehicle speeds (50, 60, 70, 80, 90 and 100 km/h). The second step is to determine the actual correction factors by dividing the coefficients in the created table (first step) with the coefficient corresponding to a water film depth of 0 mm. In the third step, a regression analysis has to be performed in order to determine the value of the parameters of the following equation:

$$\mathbf{CFW_{predicted} = 1 + a_1 \cdot log(WFD) + a_2 \cdot [V_{ref.}]^{b1} + a_3 \cdot (log[WFD] \cdot [V_{ref.}]^{b1})}$$

(equation 25)

<u>Where:</u>

$CFW_{predicted}$:	Correction factor friction due to water film depth (-)
WFD:	Water film depth (mm)
$V_{ref.}$:	Reference vehicle speed (= V - 50 km/h)
V:	Vehicle speed (km/h)
a_1, a_2, a_3, b_1:	Parameters (-)

Finally, in the fourth step, the $CFW_{predicted}$ will be determined by using equation 25. This is done in order to determine the coefficient of determination (r^2) of the linear regression line and the standard errors.

- **First step: Create table with coefficients (see table 5-7).**

Table 5-7: Locked braking force coefficients of radial tires.

	Velocity (km/h)					
t (mm)	**50 km/h**	**60 km/h**	**70 km/h**	**80 km/h**	**90 km/h**	**100 km/h**
0	0.52	0.47	0.43	0.42	0.41	0.40
1	0.52	0.47	0.42	0.37	0.29	0.20
2	0.52	0.46	0.41	0.35	0.25	0.16
5	0.52	0.46	0.40	0.33	0.22	0.12
10	0.51	0.46	0.40	0.32	0.20	0.11

- **Second step: Determine the actual correction factors (see table 5-8).**

Table 5-8: Actual correction factors CFW.

	Velocity (km/h)					
t (mm)	**50 km/h**	**60 km/h**	**70 km/h**	**80 km/h**	**90 km/h**	**100 km/h**
0	1.00	1.00	1.00	1.00	1.00	1.00
1	1.00	1.00	0.98	0.88	0.71	0.50
2	1.00	0.98	0.95	0.83	0.61	0.40
5	1.00	0.98	0.93	0.79	0.54	0.30
10	0.98	0.98	0.93	0.76	0.49	0.28

- **Third step: Linear regression analysis**

By using Microsoft Excel a regression analysis can be performed in order to determine the value of the parameters a_1, a_2, a_3, b_1 (see equation 25). After running the regression analysis, this resulted in the following values for the previously mentioned parameters.

$a_1 = -0.00347$
$a_2 = -4.2 \cdot 10^{-5}$
$a_3 = -2.689 \cdot 10^{-5}$
$b_1 = 2.4$

- **Fourth step:** Determine the $CFW_{predicted}$, r^2 of the linear regression and the standard errors.

Table 5-9: Predicted correction factors $CFW_{predicted}$.

	Velocity (km/h)					
t (mm)	**50 km/h**	**60 km/h**	**70 km/h**	**80 km/h**	**90 km/h**	**100 km/h**
0	-	-	-	-	-	-
1	1.00	0.99	0.94	0.85	0.71	0.50
2	1.00	0.99	0.93	0.82	0.65	0.40
5	1.00	0.98	0.92	0.79	0.57	0.27
10	1.00	0.98	0.91	0.76	0.52	0.18

In figure 5-3, the predicted correction factors ($CFW_{predicted}$) are plotted against the actual correction factors presented in table 5-8. The coefficient of determination r^2 reads 0.996. This indicates that the data points fit the regression line very well.

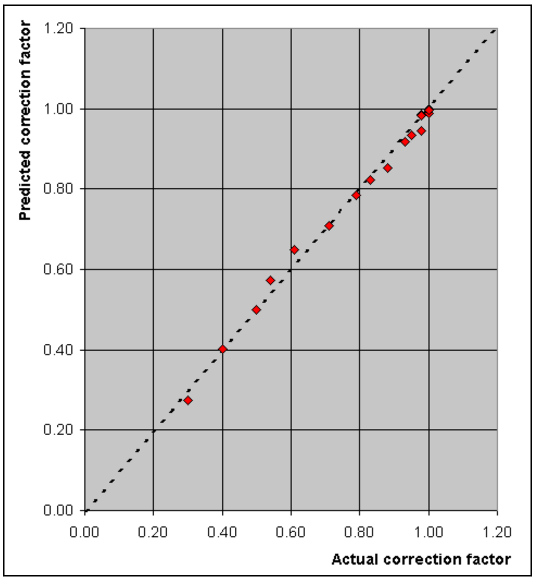


Figure 5-3: Regression line between actual correction factors and predicted correction factors.

The final equation to determine the correction factor friction due to water film depth reads:

$$\mathbf{CFW_{predicted} = 1 + -0.00347 \cdot \log(WFD) + -4.2 \cdot 10^{-5} \cdot [V-50]^{2.4} + -2.689 \cdot 10^{-5} \cdot (\log[WFD] \cdot [V-50]^{2.4})}$$

(equation 26)

This equation will be inserted in the final developed model to determine the correction factor. Further, in order to determine the reduced skid resistance, the predicted correction factor friction due to water film depth should be multiplied with the present skid resistance of the road surface with the corresponding water film layer. The crash rate that corresponds with the calculated reduced skid resistance can be determined by using equation 21 or 22.

- **Texture depth vs. Fuel consumption**

Fuel consumption [l/100 km] = 0.1818 * TD [mm] + 6.582

- **IRI (roughness) vs. Fuel consumption**

Fuel consumption [l/100 km] = 0.0386 * IRI [m/km] + 6.400

- **Texture depth vs. Traffic noise**

Noise level [dBA] = 1.143 · TD [mm] + 75.93	DAC & SMA
Noise level [dBA] = 74.9 + 0.29 * t (age of wearing course)	PAC

5.4 Calculation of the Expected Number of Crashes

In order to calculate the expected number of crashes (per year) for a road that will be designed, the (expected) average traffic volume (AADT) for that road must be known. First of all the AADT should be converted into 100 million vehicles km per year. This can be done by firstly multiplying the AADT with 365 (days) and the distance or length of the designed road (km) and at last by dividing it by 100 million. This is given with the following equation:

AADT · length of the road [km] · 365 / 100 million = number of 100 Mvhc·km/year

(equation 27)

To calculate the expected number of crashes per year the crash rate obtained from the previously mentioned relationships between road surface properties and crash rate should be multiplied with the calculated number of 100 Mvhc km/year (equation 27). So the expected number of crashes per year can be calculated with the following equation:

100 Mvhc·km/year · crash rate = Number of crashes per year *(equation 28)*

6. RESULTS AND ANALYSIS

The previous two chapters describe which databases are going to be used for the development of the final model and the research method. Furthermore, the procedure of how the data will be processed was also described. This chapter presents the results from the processed data. The results are presented in the form of in total 23 mini-models that will be collated to the final model. At first, the progression of skid resistance will be evaluated. Secondly, the progression of longitudinal unevenness or roughness will be addressed in section 6.2. Finally, the progression of rutting and texture depth (MPD) will be dealt with in sections 6.3 and 6.4.

6.1 Progression of Skid Resistance

In this section the progression of skid resistance will be addressed mainly by looking at the polishing rate of different mineral aggregates used in wearing courses of PA, SMA and DAC.

6.1.1 Motorway A1

The core of the data was obtained from measurements carried out on the Dutch motorway A1. This motorway has a porous asphalt (PA) wearing course, like almost all other roads of the motorway network in the Netherlands. Three mineral aggregates were tested on a section of this motorway to determine the polishing rate of these aggregates. The three mineral aggregates under test were porphyry, greywacke and Dutch stone (crushed gravel / Nederlandse steenslag) (see figure 6-3). For each of these mineral aggregates the Polished Stone Value (PSV) was measured in the laboratory. The PSV's of the tested aggregates are presented in table 6-1. This table shows that greywacke has the largest PSV (and consequently the highest resistance to polishing), whereas porphyry has the smallest PSV.

The skid resistance was measured on this motorway during a period covering the years 1992-1999. By plotting the measured skid resistance values against time (year of measurement) it will be possible to determine a trend line showing how fast the skid resistance increases or decreases within the years of measurement. Figure 6.1 shows the progression of the skid resistance of the three tested mineral aggregates.

Table 6-1: PSV of tested mineral aggregates in (PAC, Dutch motorway A1).

Mineral aggregate	PSV
Porphyry	52
Greywacke	60
Dutch stone chips	54

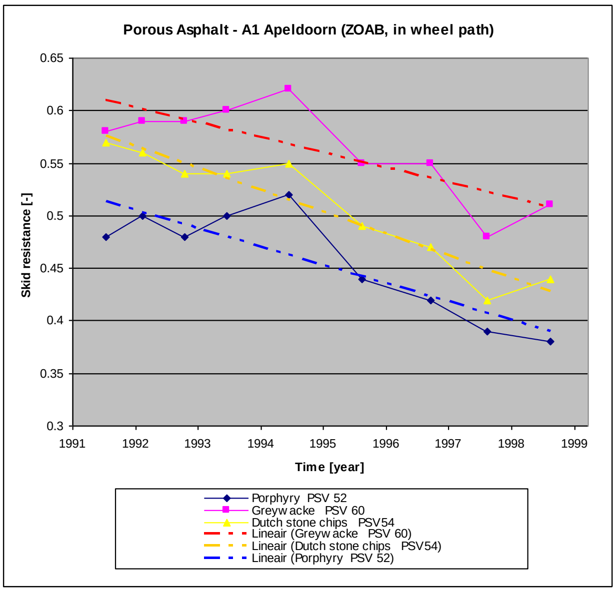


Figure 6-1: Progression of skid resistance on test sections A1.

Figure 6.1 clearly shows that greywacke has the highest initial friction coefficient. The mineral aggregate porphyry has the lowest initial friction coefficient. From the slopes of the regression lines that have been plotted for the three mineral aggregates it can be seen that the slope of the Dutch gravel regression line is the steepest. This points at a high rate of polishing. This has of course a negative effect on the friction life of the road surface, i.e. the pavement life where skid resistance will not drop below the acceptance level.

In the Netherlands, road pavement structures are built to last at least 20 years before maintenance and/or reconstruction is needed. The design life of the wearing course is usually the same for DAC wearing courses but (much) shorter for open graded wearing courses. This implies that the skid resistance should stay above the acceptance level during the design life of the wearing course.

The minimum skid resistance level accepted in the Netherlands (for the test method of 86% retarded wheel) is 0.38. During the friction life, the friction coefficient must remain above this minimum value. Figure 6-2 shows the friction life for the three mineral aggregates and also indicates how many times maintenance is required during a period of 20 years. When the level of skid resistance reaches the value of 0.38, maintenance for the wearing course is required. The graph shows that both for the mineral aggregates porphyry and the Dutch gravel, maintenance is required twice during a period of 20 years. For the mineral aggregate greywacke maintenance is required only once. It is assumed that when maintenance is carried out to the road surface, the same mineral aggregate is used once again and that the condition of the road surface starts at the same level of initial skid resistance as before. More information that can be obtained from this figure is given in table 6-2. The penultimate column of table 6-2 presents the remaining friction life after 20 years. The remaining friction life is called the salvage. From the information that can be obtained from table 6-2 the initial friction coefficient (column 3) and the polishing rate (column 4) are the most important data for the development of the final model.

The decline of skid resistance with time is typical for the investigated road section. At a later stage, the effect of traffic intensity on the rate of decline of friction will be investigated and modelled.

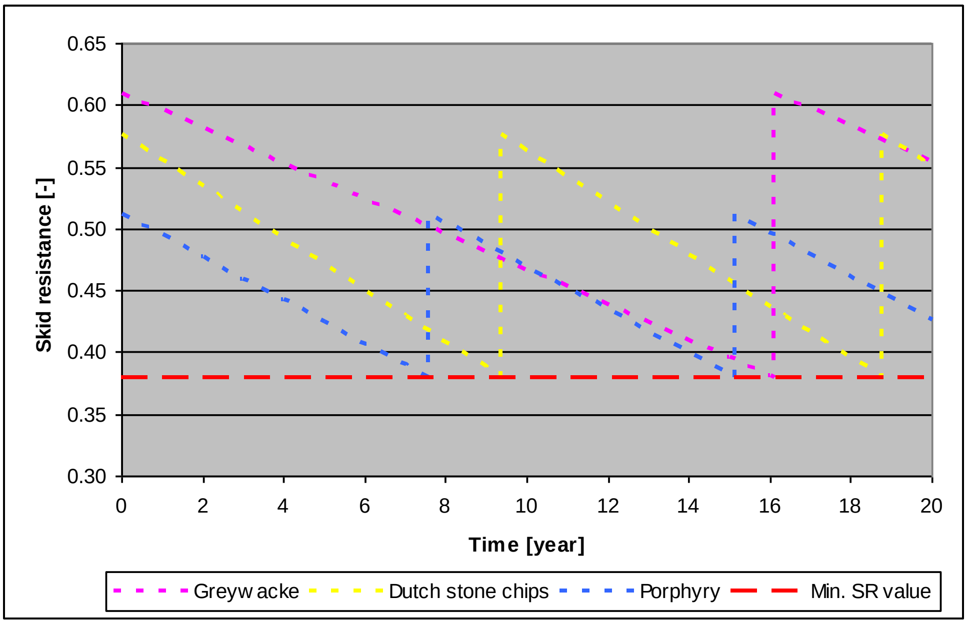


Figure 6-2: Progression of skid resistance during a period of 20 years (regression lines).

Table 6-2: Initial friction coefficient, polishing rate, friction life, x times maintenance and salvage (A1, PAC surface layer).

	PSV	Initial friction coefficient [-]	Polishing rate [per year]	Friction life [year]	x times mainte-nance	Salvage [year]	Average friction during 20 years
Porphyry	52	0.51	-0.0175	7.6	2	2.8	0.438
Greywacke	60	0.61	-0.0143	16.1	1	12.2	0.500
Dutch gravel	54	0.58	-0.0209	9.4	2	8.2	0.475

From table B-1 in Appendix B it can be seen that the mineral aggregate greywacke in porous asphalt leads to a higher average skid resistance during 20 years and will correlate to a lower average crash rate. Besides this safety aspect, greywacke has also a durable and environmental benefit that only one cycle of maintenance is required during a period of 20 years. The mineral aggregate porphyry has the poorest performance of the three mineral aggregates.

Figure 6-3: Mineral aggregates: Moraine, Dutch gravel, Porphyry and Greywacke.

6.1.2 Bamberg

In this section the development of the skid resistance on a Stone Mastic Asphalt (SMA) surface layer is addressed by studying the behaviour of different mineral aggregates. The skid resistance measurements performed on a road section in Bamberg, Germany were analysed for retrieving data on trend lines. This road section was divided in several sections where in each section another mineral aggregate was used in the SMA surface layer. The mineral aggregates used in this test were:

- Greywacke;
- Moraine split 2;
- Diabase;
- Basalt;
- Moraine split 1;
- Dolomite.

Moraine split 1 refers to Bavarian moraine and moraine split 2 is the group name for crushed gravel from borrow pit boulders from the basin of the Rhine in South Germany. Each of the six tested mineral aggregates is characterised by its own PSV. Table 6-3 presents the PSV values of the tested mineral aggregates.

Table 6-3: PSV values of the mineral aggregates (Bamberg).

Surface layer	Mineral aggregate	PSV
SMA 0/11	Greywacke	59
	Moraine split 2	53
	Diabase	55
	Basalt	47
	Moraine split 1	44
	Dolomite	40

The results of the data processed from the measurements performed on the road section in Bamberg are shown in the figures 6-4 and 6-5. The progression of the skid resistance can be described with a linear function or a power function. Tests performed in the laboratory show that the progression of the polishing of mineral aggregates can be described better with a power function. Figure 6-4 shows the progression of the skid resistance described with a linear function whereas figure 6-5 shows the trend expressed in a power function. The following equation is used for the linear function:

Linear function: $\mathbf{SR = c + (b * t)}$ *(equation 29)*

<u>Where:</u>

SR = skid resistance [-]
c = initial value of SR [-]
b = annual change of SR [-/year]
t = time [year]

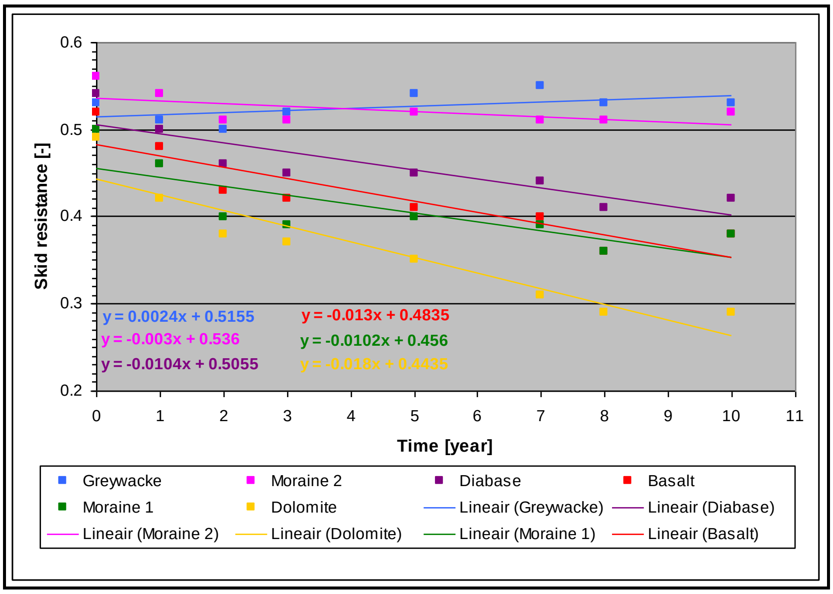


Figure 6-4: Progression of skid resistance on SMA surface layer with different mineral aggregates.

For the power function is the following equation used:

Power function: $\mathbf{SR = c * (t + 1)^b}$ *(equation 30)*

<u>Where:</u>

SR = skid resistance [-]
c = initial value of SR [-]
b = model coefficient [-]
t = time [year]

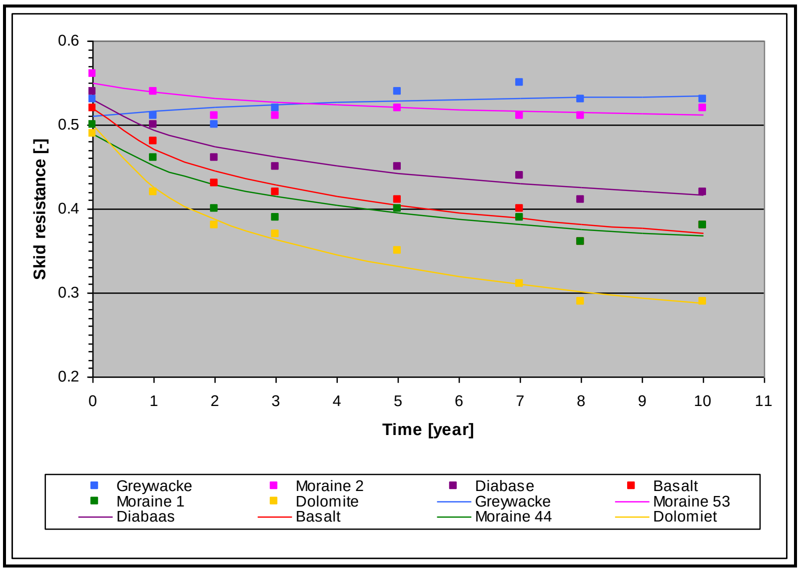


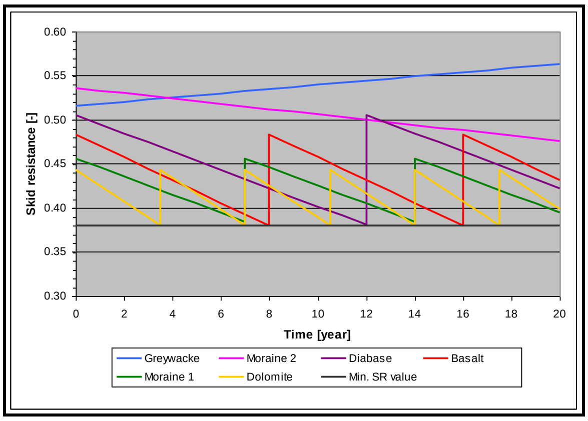
Figure 6-5: Progression of skid resistance on SMA surface layer with different mineral aggregates.

Table 6-4 presents the c and b values of the linear and power functions for the six mineral aggregates.

Table 6-4: Evaluated c and b values for the different mineral aggregates (power and linear function).

	Mineral aggregates		**Power function**			**Linear function**		
	Type	PSV	c	b	SR	c	b	SR
SMA 0/11	Greywacke	59	0.51	+0.02	$0.51 * (t + 1)^{+0.02}$	0.52	0.0024	0.0024 * t + 0.5155
	Moraine 2	53	0.55	-0.03	$0.55 * (t + 1)^{-0.03}$	0.54	-0.003	-0.003 * t + 0.536
	Diabase	55	0.53	-0.10	$0.53 * (t + 1)^{-0.10}$	0.51	-0.0104	-0.0104 * t + 0.5055
	Basalt	47	0.52	-0.14	$0.52 * (t + 1)^{-0.14}$	0.48	-0.013	-0.013 * t + 4835
	Moraine 1	44	0.49	-0.12	$0.49 * (t + 1)^{-0.12}$	0.46	-0.0102	-0.0102 * t + 0.456
	Dolomite	40	0.5	-0.23	$0.50 * (t + 1)^{-0.23}$	0.44	-0.018	-0.018 * t + 0.4435

If it is assumed that the polishing rate of the mineral aggregates was measured at the same speed that is common for a specific road, the friction life of that road can be calculated (see figures 6-6 and 6-7). These figures also show how many maintenance cycles will be needed to keep the friction level at an acceptable level.



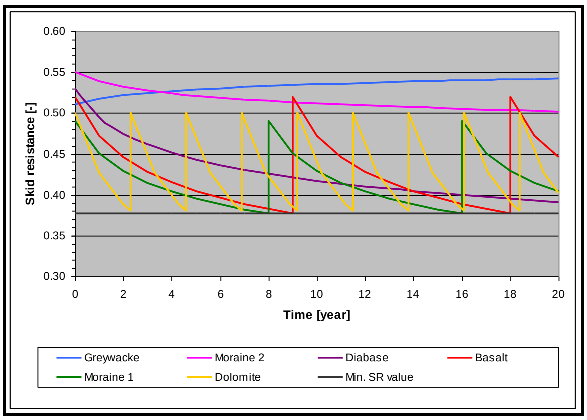


Figure 6-6: Progression of skid resistance during a period of 20 years (linear function).
Figure 6-7: Progression of skid resistance during a period of 20 years (power function).

Tables 6-5 and 6-6 display the calculated friction life for the difference mineral aggregates. They also present the number of maintenance cycles and the salvage period.

Table 6-5: Initial Friction life, x times maintenance and salvage calculated with linear function (Bamberg, SMA surface layer).

		Friction life [year]	x times maintenance	Salvage [year]	Average friction during 20 years
SMA 0/11	Greywacke	20	0	20	0.545
	Moraine 2	20	0	20	0.509
	Diabase	12	1	4	0.451
	Basalt	8	2	4	0.427
	Moraine 1	7	2	1	0.421
	Dolomite	3.5	5	1	0.397

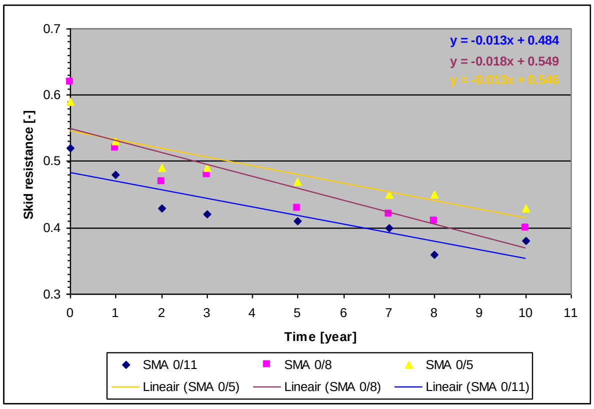
Table 6-6: Initial Friction life, x times maintenance and salvage calculated with power function (Bamberg, SMA surface layer).

		Friction life [year]	x times maintenance	Salvage [year]	Average friction during 20 years
SMA 0/11	Greywacke	20	0	20	0.533
	Moraine 2	20	0	20	0.516
	Diabase	20	0	7	0.428
	Basalt	9	2	7	0.430
	Moraine 1	8	2	4	0.420
	Dolomite	2.3	8	0.7	0.425

From table B-2 in Appendix B it can be seen that the mineral aggregate greywacke in stone mastic asphalt concrete offers a higher average skid resistance during 20 years and correlates automatically with a lower average crash rate. Besides this, with the use of the mineral aggregate greywacke zero times maintenance is required during the period of 20 years. The mineral aggregate dolomite has the poorest performance of the six mineral aggregates. With the use of this mineral aggregates five times maintenance are required during the period of 20 years (see table B-3 in Appendix B).

<u>Basalt in SMA and DAC (different grading) Bamberg</u>

In this part the development of the skid resistance on Stone Mastic Asphalt (SMA) and Dense Asphalt Concrete (DAC) surface layers with different grading will be determined. This is done to see what effect the grading has on the polishing rate of the mineral aggregate Basalt. Measurements were performed on three SMA surface layers in which each surface layer has a different grading, i.e. SMA 0/11, SMA 0/8 and SMA 0/5 mixture. This was also done for three DAC layers in which each surface layer has again its own specific grading. The PSV of the mineral aggregate Basalt used in the previously mentioned surface layers is 47. After processing the data with the help of Microsoft Excel this resulted in the following four figures 6-8 to 6-11. In figures 6-8 and 6-9 the development of the skid resistance for the SMA surface layers is given and in figures 6-10 and 6-11 that of the DAC surface layers. The development of the skid resistance is once again described with a linear and a power function.



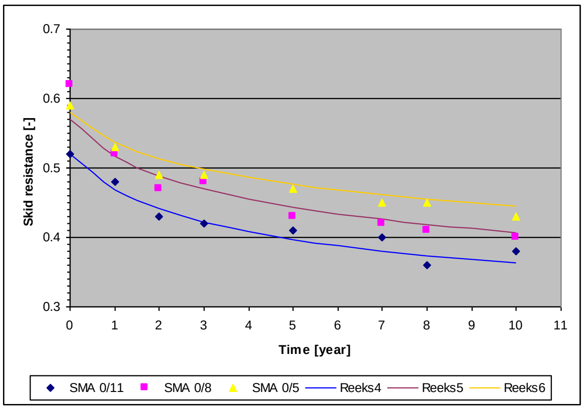
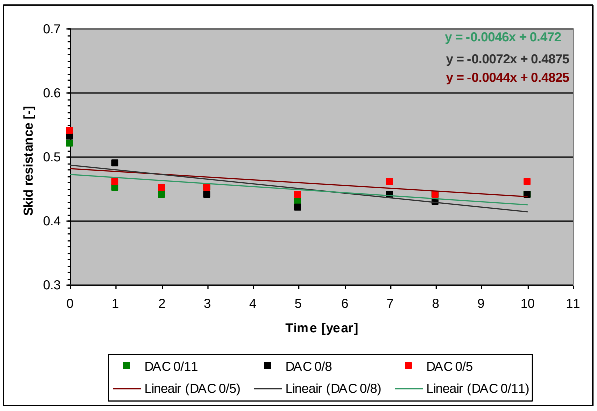


Figure 6-8: Development of skid resistance for SMA surface layers with different grading (left linear).
Figure 6-9: Development of skid resistance for SMA surface layers with different right power).



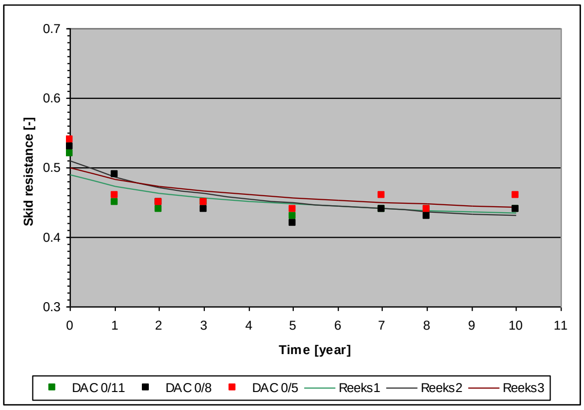


Figure 6-10: Development of skid resistance for DAC surface layers with different grading (left linear).
Figure 6-11: Development of skid resistance for DAC surface layers with different grading (right power).

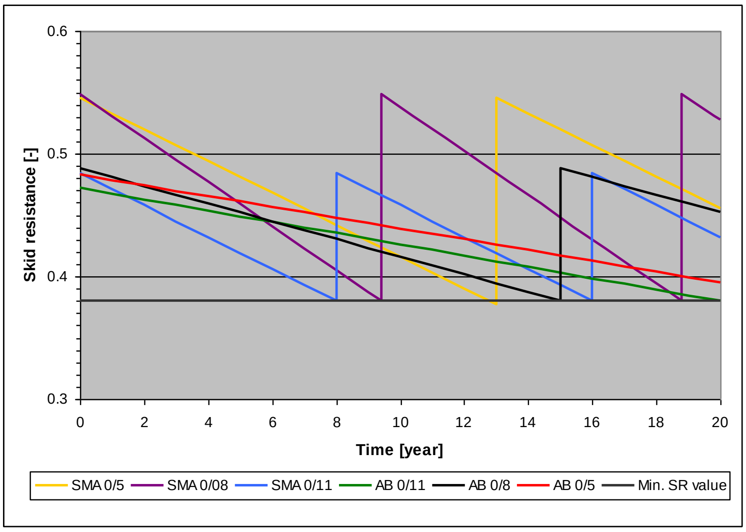


Figure 6-12: Expected linear development of skid resistance for SMA and DAC surface layers with different grading (Basalt).

Table 6-7: Friction life, average skid resistance and average crash rate (approximated with a linear function).

	Top layer & Grading	Friction life [year]	x times maintenance	Salvage [year]	Average skid resistance	Average crash rate
Basalt	SMA 0/11	8	2	4	0.431	10.808
	SMA 0/8	9.5	2	8.5	0.455	9.486
	SMA 0/5	13	1	6	0.469	8.429
	DAC 0/11	20	0	0	0.424	11.339
	DAC 0/8	15	1	10	0.439	10.203
	DAC 0/5	20	0	3	0.436	10.293

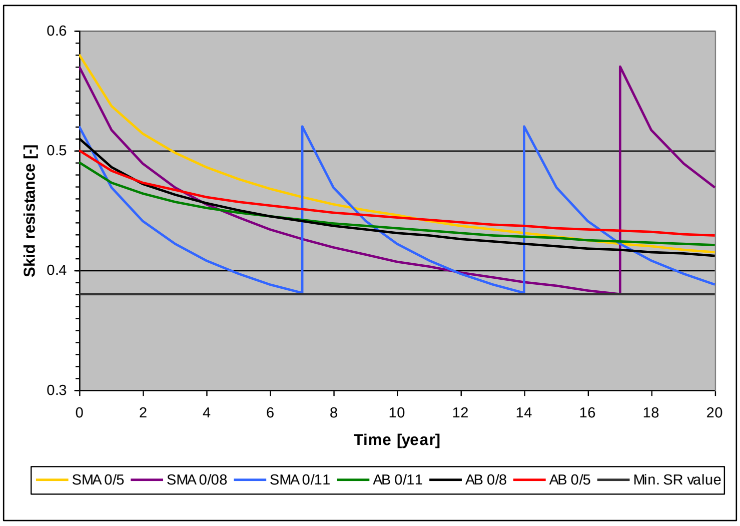


Figure 6-13: Expected power development of skid resistance for SMA and DAC surface layers with different grading (Basalt).

Table 6-8: Friction life, average skid resistance and average crash rate (approximated with a power function).

	Top layer & Grading	Friction life [year]	x times maintenance	Salvage [year]	Average skid resistance	Average crash rate
Basalt	SMA 0/11	7	2	1	0.430	11.095
	SMA 0/8	17	1	14	0.447	10.151
	SMA 0/5	20	0	20	0.458	8.970
	DAC 0/11	20	0	20	0.440	9.901
	DAC 0/8	20	0	20	0.439	10.063
	DAC 0/5	20	0	20	0.449	9.255

In the Netherlands, the mineral aggregate in wearing courses must have at least a PSV of 58 (for motorways). The used mineral aggregates should also have 0% round surface. In the Netherlands, a distinction is made between three types of stone aggregates (type 1/2/3). Each type of stone aggregate is prescribed for a particular traffic class. For major main roads and provincial roads the application of stone aggregates type 3 is required. This is only needed for discontinuous coarse graded mixtures. PAC and SMA are discontinuous graded asphalt mixtures and DAC is a continuous graded asphalt mixture. So for SMA or PAC mixtures the application of stone aggregates type 3 is required.

The resistance to polishing and the sharpness of the stone aggregates are very relevant with respect to the traffic safety. If the micro texture of the stone aggregates is polished by the traffic, this leads to a reduction of the skid resistance. The sharpness (micro texture) of the stone aggregates is important to break through the water film on the surface layer. For a durable skid resistance performance a good surface drainage and a polishing resistant aggregate are needed.

The most widely used mineral aggregates in the Netherlands in wearing courses are Greywacke, Moraine, Porphyry, Bestone and Dutch gravel. Ducth gravel is not used anymore in PAC surface layers because it doesn't belong to the stone type 3 anymore. Phorphyry is also not used anymore due to its unfavourable polishing behaviour. The mineral aggregates Greywacke and Moraine come from Germany (Steinbruch Berge and Rhine). The Dutch gravel comes from the Maas river and Bestone comes from Norway (Norwegian gravel). It is very important to know the place where these mineral aggregates come from. The transport distance also determines how environment-friendly the mineral aggregates are. Larger transport distances lead to more fuel consumption. It must be said that the most aggregates are transported by ships. This is also the reason why the most asphalt concrete plants are established close to water.

6.1.3 N732 Lonneker - Losser

On the road section Lonneker-Losser skid resistance measurements were performed to study the performance of the mineral aggregate greywacke (PSV 61) in a SMA wearing course and moraine (PSV 53) in a DAC and PAC wearing course. The performance of these tested mineral aggregates can be seen in figure 6-14. In this study, the phase that indicates the removal of the bitumen film from the surface aggregates will not be taken into consideration (NL: aanvangstroefheid). The maximum values for the skid resistance of the mineral aggregates are taken to be the initial level of skid resistance for the tested aggregates (see figure 6-15). The performance of the mineral aggregates illustrated in figure 6-15 are described with linear equations. These equations will be used to describe the performance of the tested aggregates in the final model.

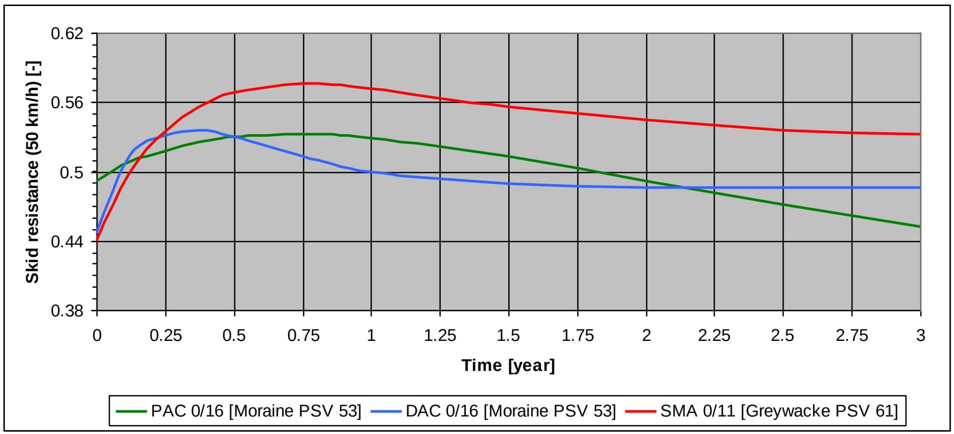


Figure 6-14: Skid resistance measurements Lonneker-Losser.

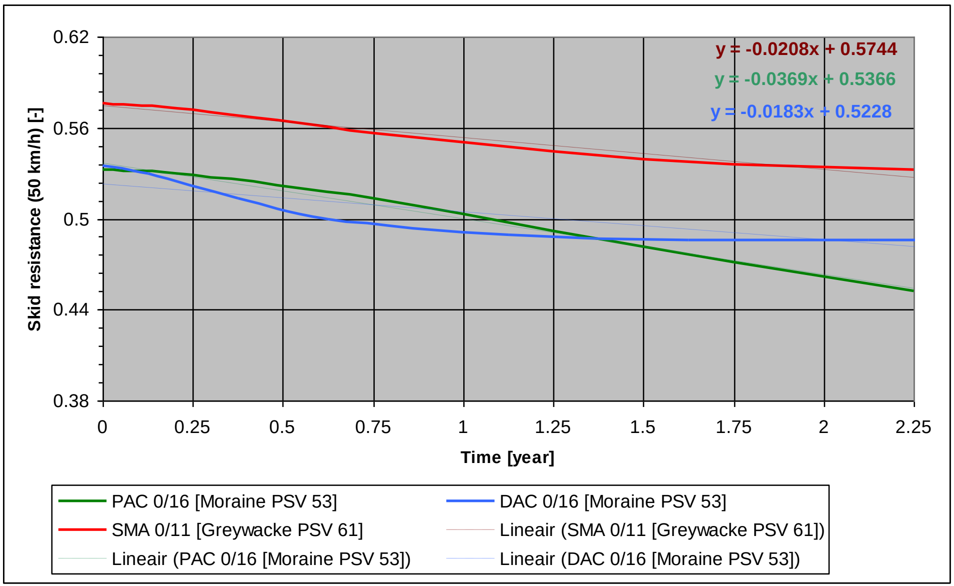


Figure 6-15: Skid resistance measurements Lonneker-Losser without the phase removal of the bitumen film from the surface aggregates.

Although earlier it was mentioned that tests performed in the laboratory show that the progression of the polishing of mineral aggregates better can be described with a power function, it is decided to still use the linear progression for the development of the skid resistance due to simplicity of further calculation and development of the final model. The service life (time when maintenance is required) can be calculated much easier with the use of linear progressions.

6.2 Modeling development of roughness

Longitudinal unevenness has a major influence on the driving comfort of the road users. Severe longitudinal unevenness has not only a negative influence on the driving comfort but also on the traffic safety. The development of longitudinal unevenness is mainly caused by unequal settlements in the subgrade and the traffic loads. Besides unequal settlements and traffic loads also the asphalt thickness has an effect on the progression of longitudinal unevenness. Settlements in the subgrade occur mainly in subgrades like clay and peat. These subgrades have a low bearing capacity. The effect of the traffic loads on the progression of longitudinal unevenness is quite logic. The progression of longitudinal unevenness on lightly loaded pavements is much slower than on heavily loaded pavements. In order to model the development of longitudinal unevenness it is chosen to make a distinction between the asphalt thickness and the type of subgrade. This will result in four mini models. Progression of unevenness on a clay subgrade for a thin and thick asphalt layer and progression of longitudinal unevenness on a sand subgrade for a thin and a thick asphalt layer. Asphalt thicknesses of 200 mm or smaller belong to thin asphalt and asphalt thicknesses greater than 200 mm to thick asphalt.

- Thin asphalt: thickness asphalt h ≤ 200 mm
- Thick asphalt: thickness asphalt h > 200 mm

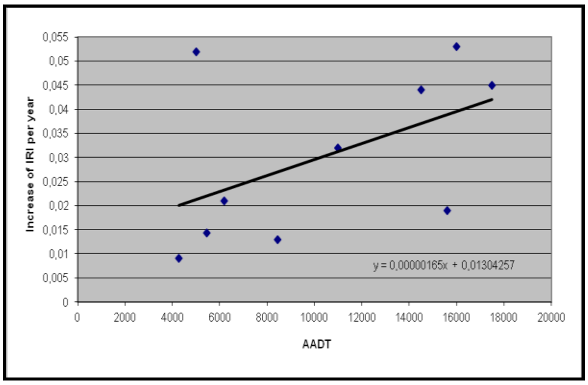
The progression of longitudinal unevenness is modeled by using data from 32 SHRP-NL road sections with different asphalt thickness and traffic intensity (see table B-4 in Appendix B). The development of longitudinal unevenness is separately analyzed for each road section. In the analysis a linear model is used. The general form of this linear model is as follows:

$$\mathbf{IRI = a + b \cdot t} \qquad \textit{(equation 31)}$$

Where:
IRI = International Roughness Index [m/km]
t = age wearing course [year]
a & b = model parameters [-]

The value of model parameter 'a' represent the initial IRI value. The value of model parameter 'b' represent the average increase of the longitudinal unevenness.

By means of regression analyses the model parameter 'b' is determined for each road section. This is the most relevant parameter of the model which once again represents the average increase of the IRI. In order to see if the traffic intensity has influence on the value of the model parameter 'b', the relation between the model parameter 'b' and the AADT is also checked. From the following figures 6-16 to 6-19 it can be seen that the AADT doesn't have a significant relation with the value of the model parameter 'b'. The spreading around three of the four models is too large. These are then not suitable to get a good indication of the annual IRI increase.
Afterwards it must be said that it was not necessary to consider the asphalt thickness and the AADT as individual variables because these variables are interrelated. In other words these variables are not independent. The higher the AADT, the thicker the asphalt layer.



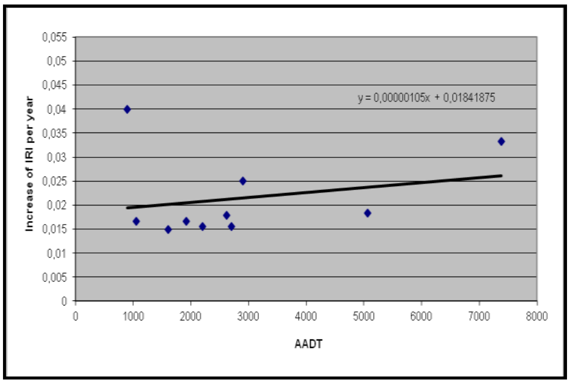
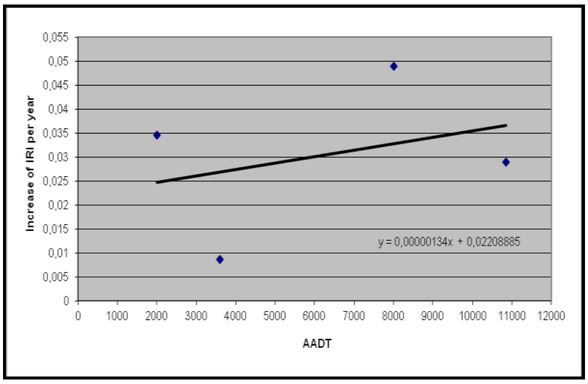


Figure 6-16: Relation between model parameter b and AADT for asphalt thickness > 200 mm on sand subgrade (left).
Figure 6-17: Relation between model parameter b and AADT for asphalt thickness ≤ 200 mm on sand subgrade (right).



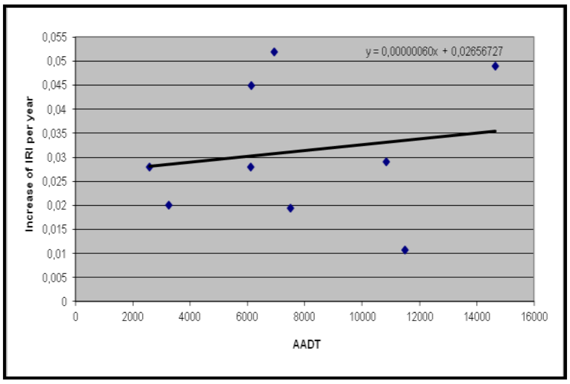


Figure 6-18: Relation between model parameter b and AADT for asphalt thickness ≤ 200 mm on clay subgrade (left).
Figure 6-19: Relation between model parameter b and AADT for asphalt thickness > 200 mm on clay subgrade (right).

Progression of longitudinal unevenness

- Mini-model 17: (subgrade = sand, asphalt thickness = thick)
  Progression function: $\boldsymbol{16.5 \cdot 10^{-7} * AADT + 0.01304}$
- Mini-model 18: (subgrade = sand, asphalt thickness = thin)
  Progression function: $\boldsymbol{10.5 \cdot 10^{-7} * AADT + 0.01842}$
- Mini-model 19: (subgrade = clay, asphalt thickness = thick)
  Progression function: $\boldsymbol{6.0 \cdot 10^{-7} * AADT + 0.02657}$
- Mini-model 20: (subgrade = clay, asphalt thickness = thin)
  Progression function: $\boldsymbol{13.4 \cdot 10^{-7} * AADT + 0.02209}$

From the analysis discussed above it can be concluded that the annual increase of the IRI has no significance relation with the AADT. So the four equations given here are not suitable to use. To still be able to use models for the development of longitudinal unevenness, the following relationships will be used (see table 6-9). These relationships are found in the CROW publication 169 [Modellen voor wegbeheer, 2002].

Table 6-9: Average annual increase of IRI for different types of pavement structures (CROW publication 169, 2002).

Elastic modulus of subgrade	Thickness asphalt ≤ 200 mm	Thickness asphalt > 200 mm
E_{mod} < 100 MPa	0.084 m/km per year	0.046 m/km per year
100 < E_{mod} < 150 MPa	0.033 m/km per year	0.029 m/km per year
E_{mod} > 150 MPa	0.027 m/km per year	0.029 m/km per year

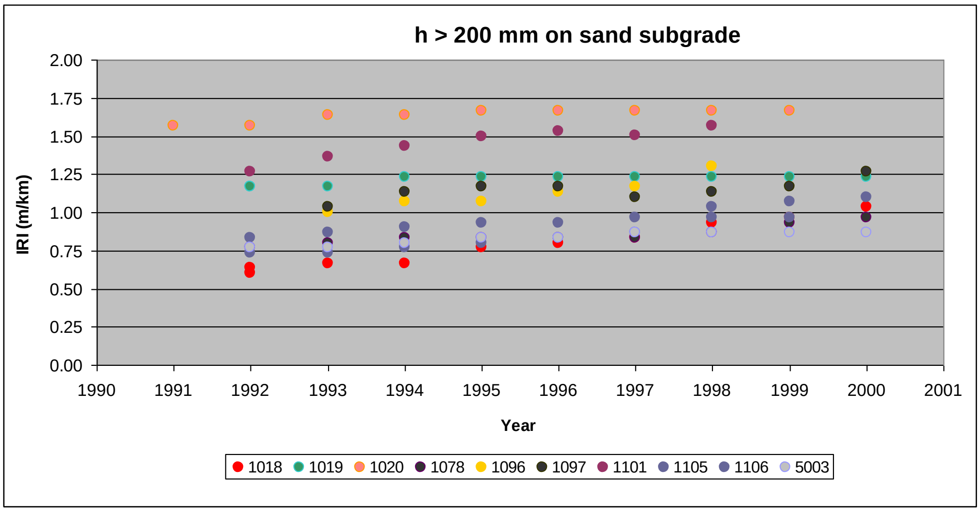


Figure 6-20: Longitudinal unevenness progression of SHRP road sections with asphalt thickness > 200 mm and on sand subgrade.

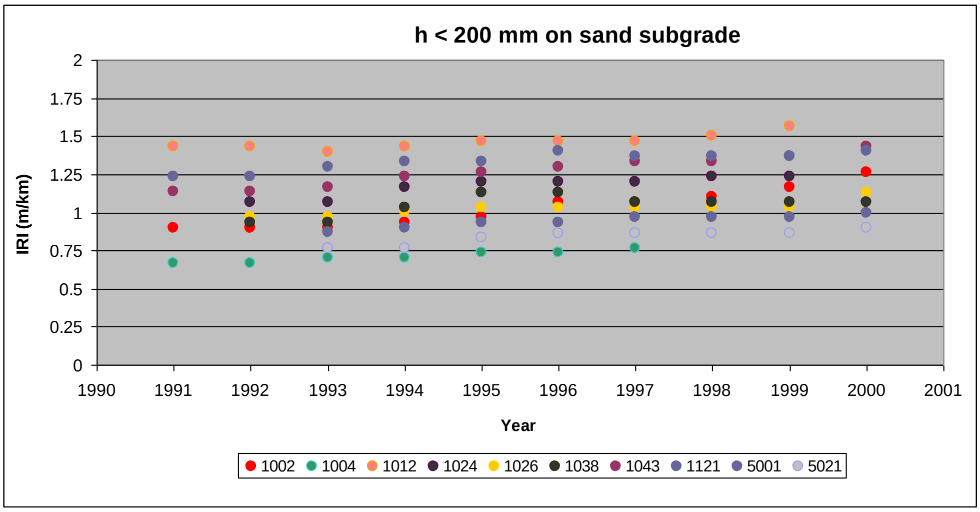


Figure 6-21: Longitudinal unevenness progression of SHRP road sections with asphalt thickness < 200 mm and on sand subgrade.

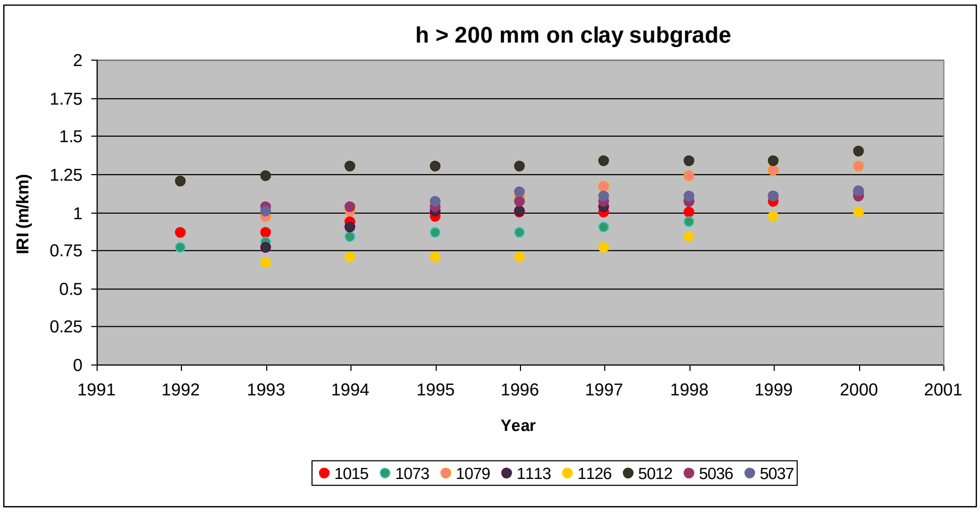


Figure 6-22: Longitudinal unevenness progression of SHRP road sections with asphalt thickness > 200 mm and on clay subgrade.

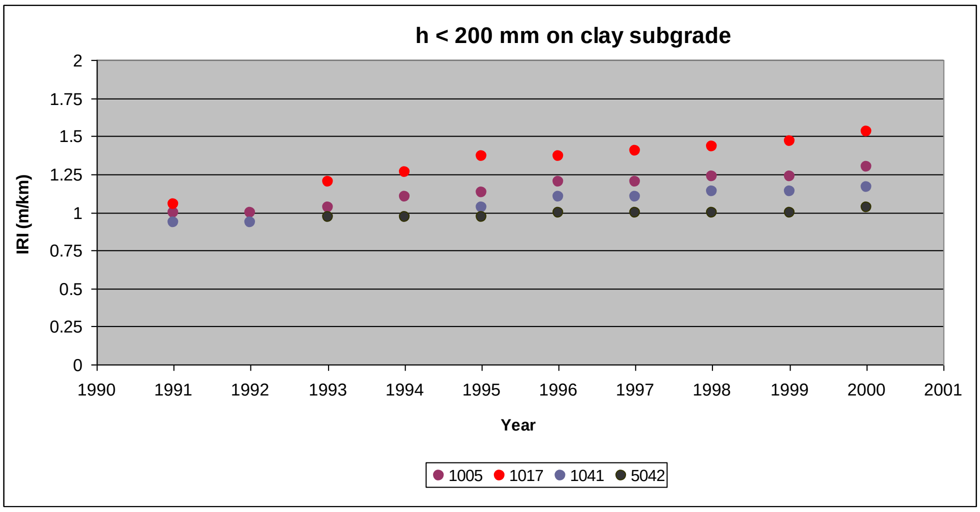


Figure 6-23: Longitudinal unevenness progression of SHRP road sections with asphalt thickness < 200 mm and on clay subgrade.

6.3 Development of Rutting

This paragraph describes the method which is used for the modeling of the development of rutting. For the analysis of the development of rutting the numbers of available data points and the ranges of observed rut depth are of great importance. The number of available data points determine the reliability with which the regression lines can be defined. The ranges of observed rut depths give an indication of the relevance of the rutting data. For this study it is chosen to use road sections where at least six data points are available. By taking road sections with more data points a better picture can be obtained for the development of rutting.

For each type of wearing course (DAC, SMA, PAC) five road sections are selected where the development of rutting will be described. To describe the development of rutting on the three types of wearing courses, a power model is used. The general form of this model is shown in the following equation.

$$\mathbf{RD = a * t^{b}} \qquad \textit{(equation 32)}$$

<u>Where:</u>

RD = rut depth [mm]
t = age wearing course [year]
a & b = model parameters [-]

By means of regression analyses the model parameters 'a' and 'b' are determined for each road section. Before the regression analyses are performed, graphs are created where the measured rut depth is plotted against the age of the wearing course. Figure 6-24 shows for the five selected road sections with a DAC wearing course how the model describes the rutting and what is the prediction of the further development of rutting. In table 6-10 the determined 'a' and 'b' values are given for each road section. Besides this, the AADT that belongs to the selected road sections is also given. This is done in order to see if the traffic intensity has an influence on the value of the model parameter 'b'. Figure 6-25 shows the correlations between the AADT and the 'a' and 'b' values. These correlations can be described with the equations given in figure 6.25 ('a' and 'b' as a function of AADT).

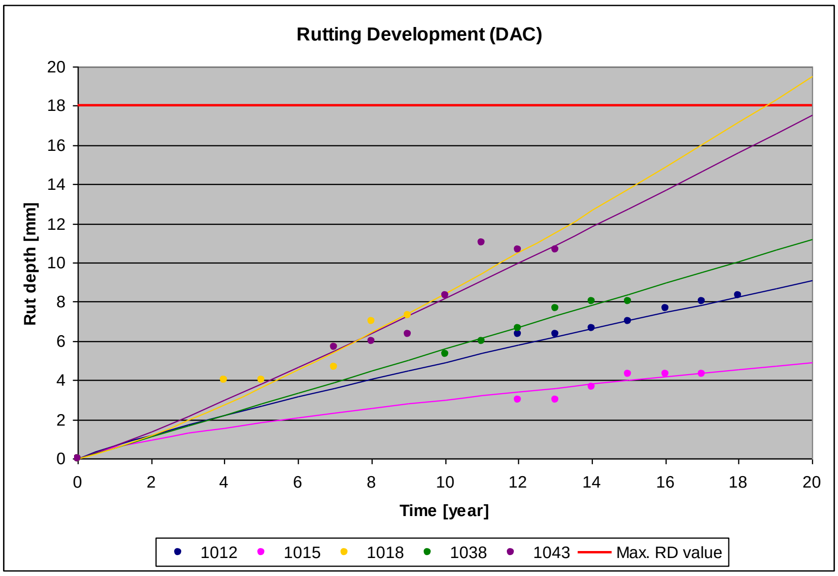


Figure 6-24: Modelling of the development of rutting by means of a power function (road sections with a DAC wearing course).

Table 6-10: AADT, a and b values for selected road sections with a DAC wearing course.

Road sections	AADT per direction (one lane)	a-value	b-value
1012	1610	0.65	0.88
1015	2590	0.6	0.7
1018	5000	0.65	1.1
1038	2700	0.56	1.0
1043	7380	0.52	1.21

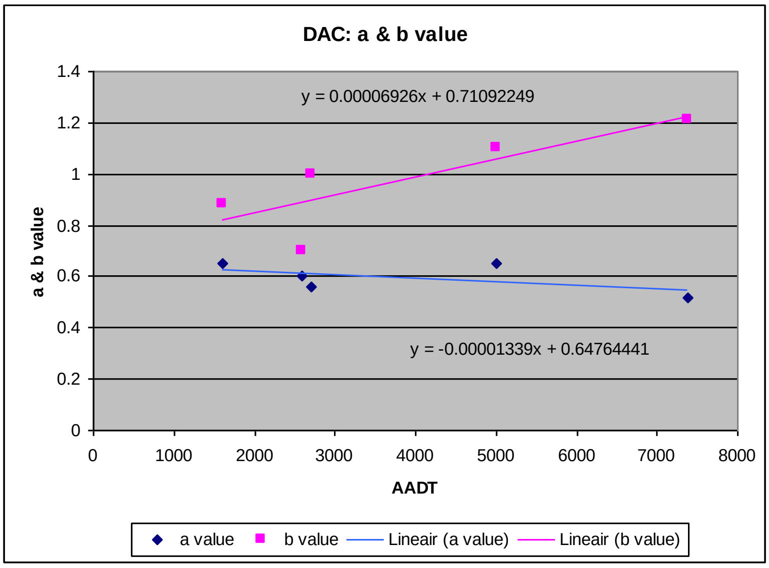


Figure 6-25: Correlation between AADT and model parameters a & b (road sections with a DAC wearing course).

For the modelling of the development of rutting for SMA and PAC wearing courses the same method is used as for the DAC. For the selected road sections with a SMA wearing course, figures 6-26 and 6-27 can be seen together with table 6-11. The development of rutting on road section 1004 (see figure 6-26) departs from the rest of the data. A possible explanation for this outlier is that we have to do with an overfilled SMA. SMA wearing courses are very sensitive to overfilling of bitumen. The data corresponding to road section 1004 has been excluded for further analysis. For the selected road sections with a PAC wearing course, figures 6-28 and 6-29 can be seen together with tables 6-11 and 6-12.

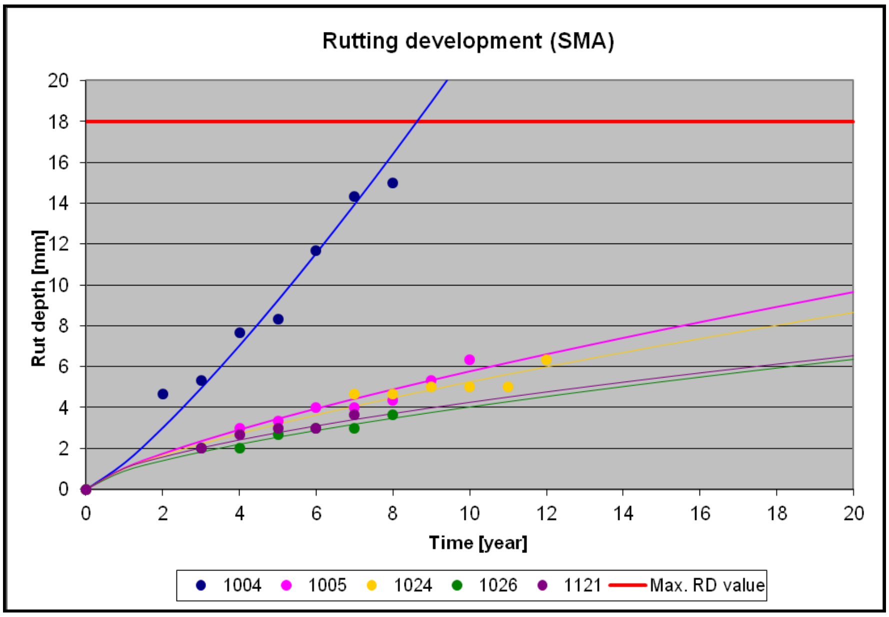


Figure 6-26: Modelling of the development of rutting by means of a power function (road sections with a SMA wearing course).

Table 6-11: AADT, a and b values for selected road sections with a SMA wearing course.

Road sections	AADT per direction (one lane)	a-value	b-value
1004	1925	1.3	1.22
1005	2000	1.05	0.74
1024	2900	1.0	0.72
1026	2200	0.88	0.66
1121	1050	1.02	0.62

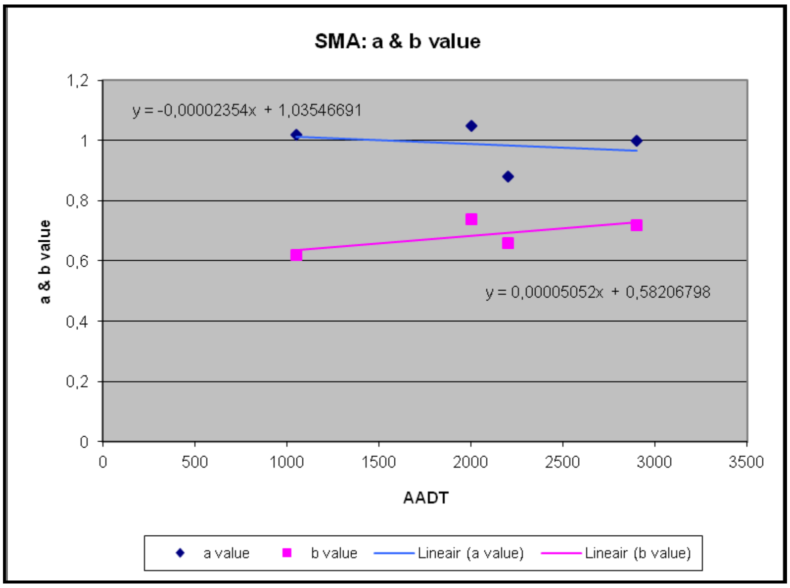


Figure 6-27: Correlation between AADT and model parameters a & b (road sections with a SMA wearing course).

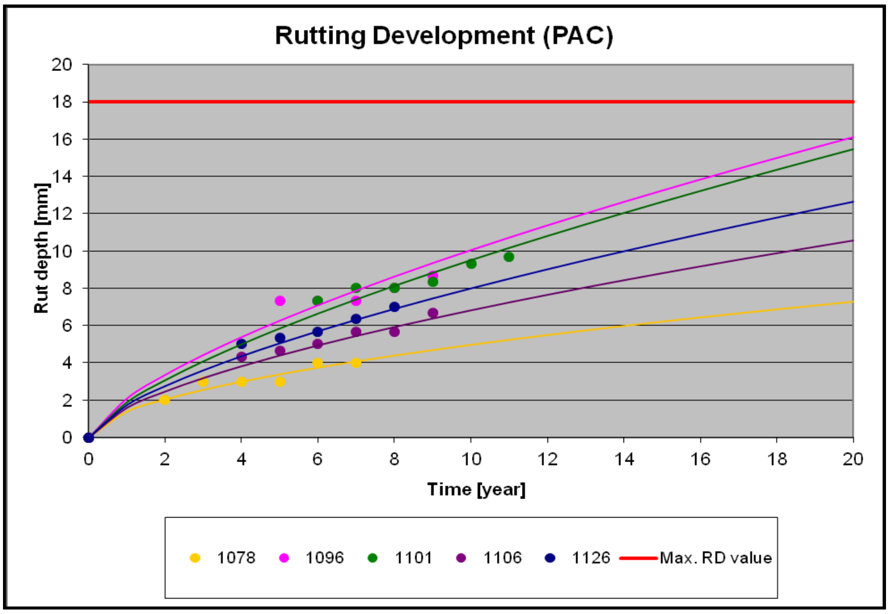


Figure 6-28: Modelling of the development of rutting by means of a power function (road sections with a PAC wearing course).

Table 6-12: AADT, a and b values for selected road sections with a PAC wearing course.

Road sections	AADT per direction (one lane)	a-value	b-value
1078	6200	1,4	0,55
1096	16000	2,1	0,68
1101	17500	1,9	0,7
1106	10994	1,6	0,63
1126	14655	1,75	0,66

The AADT presented in the tables 6-10 to 6-12 (second column) is the AADT on one traffic lane (the most heavily trafficked one).

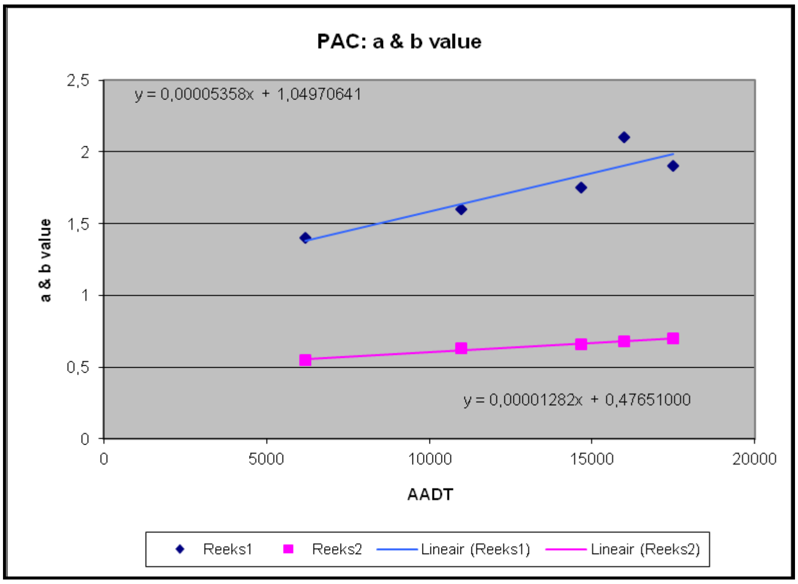


Figure 6-29: Correlation between AADT and model parameters a & b (road sections with a PAC wearing course).

By correlating the development of rutting to the age of the wearing course and the AADT the following three mini models can be inserted in the final model:

Mini model 21 (DAC): $RD = (-1{,}34 \cdot 10^{-5} * AADT + 0.648) * t^{(6.926 \cdot 10\text{-}5 * AADT + 0.711)}$

Mini model 22 (SMA): $RD = (-2{,}35 \cdot 10^{-5} * AADT + 1.035) * t^{(50.52 \cdot 10\text{-}5 * AADT + 0.582)}$

Mini model 23 (PAC): $RD = (5{,}36 \cdot 10^{-5} * AADT + 1.050) * t^{(1.282 \cdot 10\text{-}5 * AADT + 0.477)}$

6.4 Change (progression) of the Texture

To get an indication of how the texture (MPD) of road surfaces of different types of wearing course changes with the time, the measured MPD data are plotted against the time. From the results of the roads N62 Westerscheldetunnelweg (figure 6-30) and N273 Napoleonsweg (figure 6-31) it can be seen that the MPD increase with time. These two roads have a SMA wearing course. The average annual increase of the MPD for SMA wearing courses is 0.024 mm. The green line in the figures 6-30 to 6-35 indicates the regression line of the MPD data measured between the wheel paths (MPD-B). The blue line in the figures 6-30 to 6-35 indicates the regression line for the MPD-data measured in the right wheel path (MPD-R).

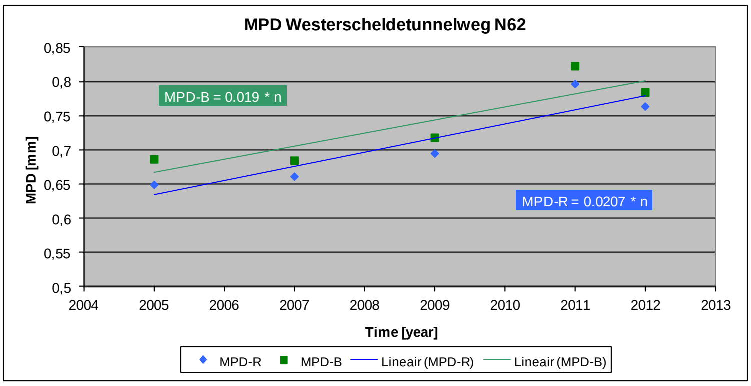


Figure 6-30: Change of MPD with time for N62, SMA wearing course.

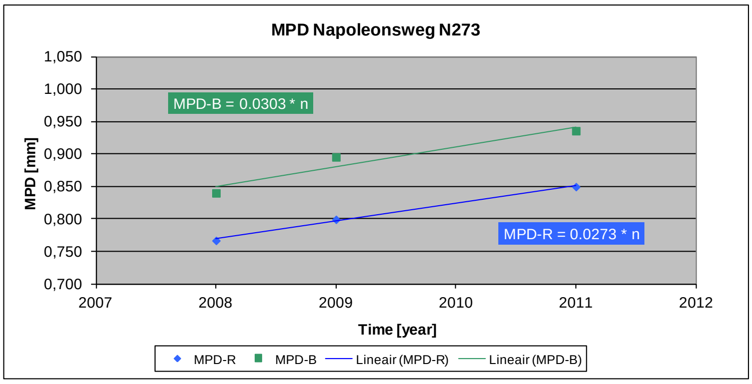


Figure 6-31: Change of MPD with time for N273, SMA wearing course.

For DAC wearing courses (Tevixhoeksweg and part of the Westerscheldetunnelweg) it can be seen that the MPD also increase with the time. The average annual increase of the MPD for DAC wearing courses is 0.034 mm. Figure 6-32 shows the progression of the MPD with time for the 'Tevixhoekweg' and figure 6-33 for the ''Westerscheldetunnelweg'.

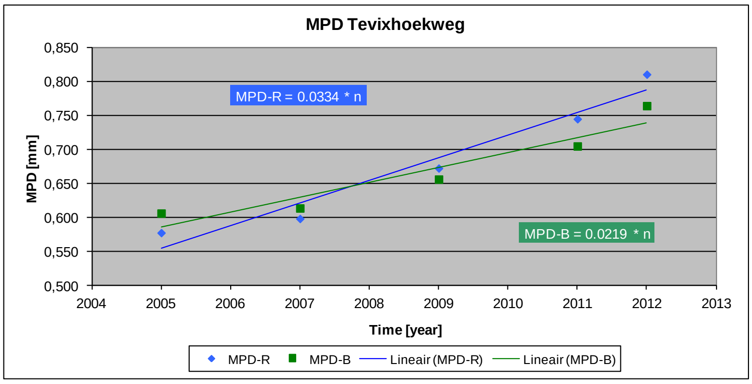


Figure 6-32: Change of MPD with time for Tevixhoekweg, DAC wearing course.

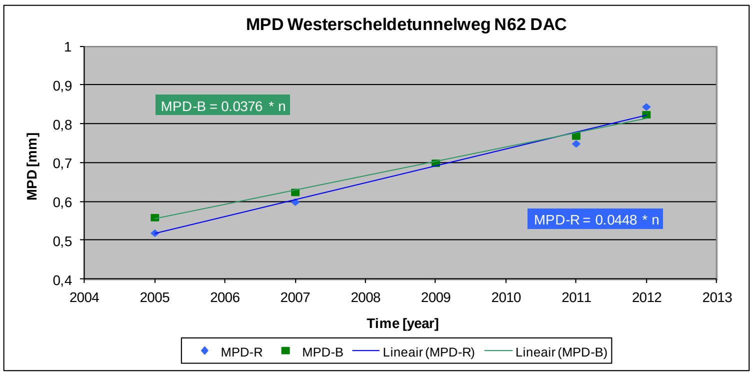


Figure 6-33: Change of MPD with time for Westerscheldetunnelweg, DAC wearing course.

In contrast to SMA and DAC wearing courses, the MPD for PAC wearing courses decreases with time. This can be seen from the two figures of the progression of the MPD for the two roads A8 and A10 (Coentunnel, see figure 6-34 and 6-35). The average annual decrease of the MPD for PAC wearing courses is -0.041 mm. A possible explanation for this decrease of the MPD for PAC wearing courses is the occurrence of raveling. This was earlier discussed in paragraph 3.2.4.2 and also shown with a sketch.

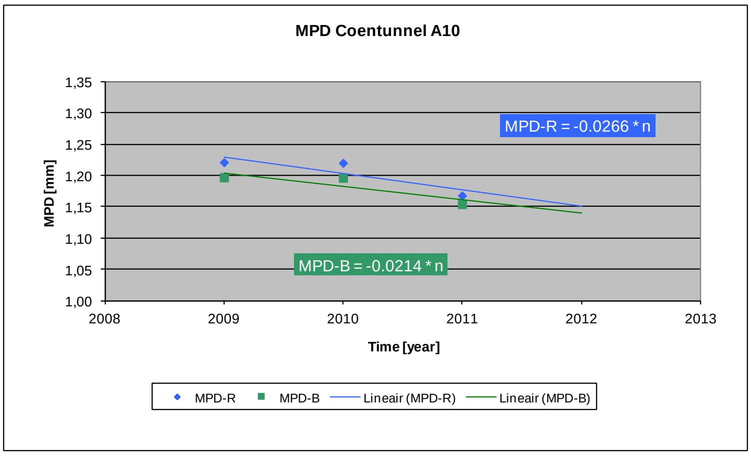


Figure 6-34: Change of MPD with time for part of A10, PAC wearing course.

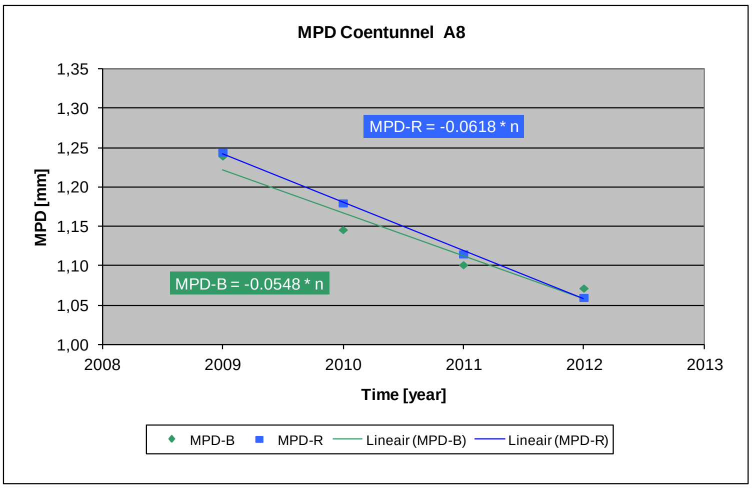


Figure 6-35: Change of MPD with time for part of A8, PAC wearing course.

Table 6-13: Range of obtained MPD values from measurement data and in the literature for the three types of wearing course.

Wearing course	MPD [mm] between wheel paths	MPD [mm] in right wheel path	MPD-range [mm] found in the Literature
SMA	0.67 - 0.80 mm	0.63 - 0.78 mm	**0.6-1.0 mm** SMA 0/8, 0/11
SMA	0.85 - 0.94 mm	0.77 - 0.85 mm	
DAC	0.58 - 0.74 mm	0.55 - 0.78 mm	**0.4-0.8 mm** DAC 0/8, 0/11, 0/16
DAC	0.56 - 0.82 mm	0.52 - 0.83 mm	
PAC	1.22 - 1.06 mm	1.24 - 1.06 mm	**1.5-1.9 mm** ZOAB 0/16
PAC	1.20 - 1.14 mm	1.23 - 1.15 mm	

Table 6-13 is given to illustrate the range of MPD values obtained from measurement data for the three different types of wearing course. In the last column of this table also MPD values are given that were found in the literature for the three previously mentioned wearing courses. From these values it can be seen that the average obtained MPD values for PAC wearing courses are rather low. The obtained average MPD values for the SMA and DAC wearing courses are in the range of the values found in the literature.

By collecting data (AADT, number of traffic lanes in one direction, percentage car and truck traffic) of the road sections, where the development of MPD is determined, the development of the MPD can be determined as a variable of the number of axle loads. This results in the following equations for the three investigated wearing courses.

Mini model 14 (PAC): $MPD = 1.7 - 6.55 \cdot 10^{-7}$ * axle loads on the heaviest trafficked lane.
Mini model 15 (SMA): $MPD = 0.8 + 1.92 \cdot 10^{-6}$ * axle loads on the heaviest trafficked lane.
Mini model 16 (DAC): $MPD = 0.6 + 4.15 \cdot 10^{-6}$ * axle loads on the heaviest trafficked lane.

The calculations that result in these equations can be seen in Appendix C.

6.5 Traffic Distribution

Traffic distribution used to calculate the decrease of skid resistance per passing axle load.

To calculate the number of axle loads and tire passes per lane assumptions were made for truck and passenger car distributions on the carriageway. Especially that data on the most nearside lane (slow lane) are important. Based on the number of trucks and passenger cars that pass on this traffic lane, the total number of axle loads can be calculated. For this calculation it was assumed that a passenger car has two axles (front and rear) and that the average number of axles for trucks is 3.5.

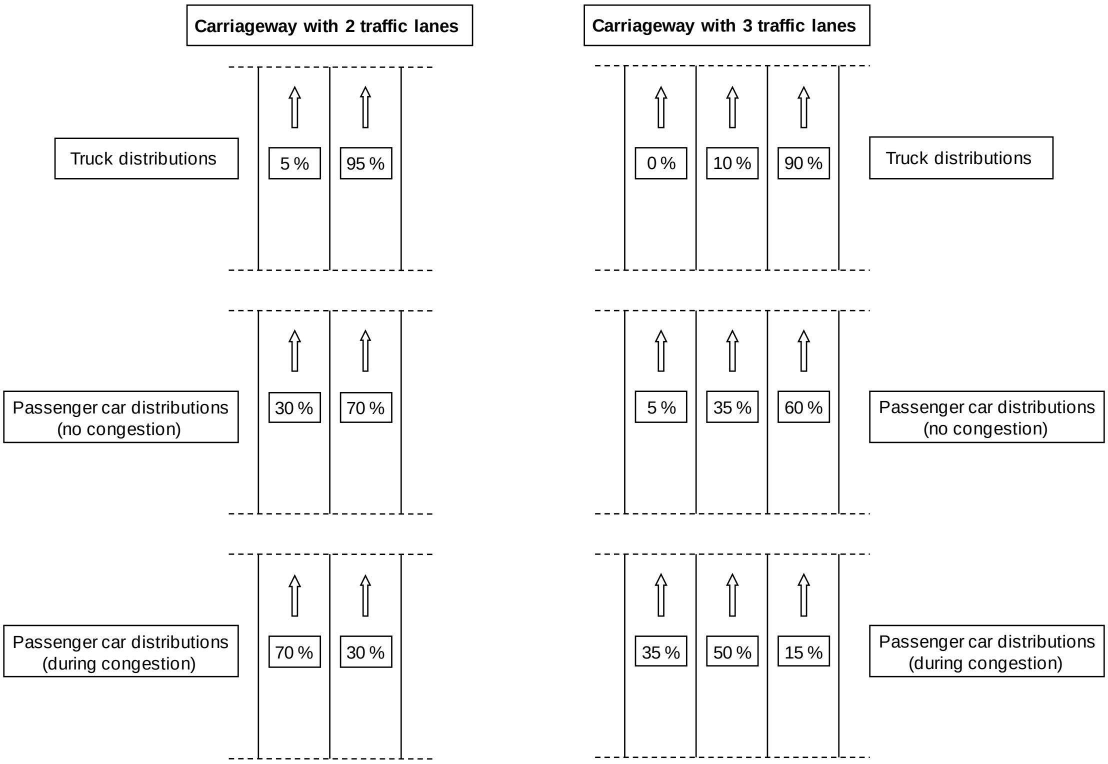


Figure 6-36: Traffic distribution.

For a carriageway that consists of two traffic lanes the truck distributions are assumed to be as follows: right lane 95% and left lane 5% (see figure 6-36). This distribution is also used in the Dutch structural pavement design procedures.

During congestion it is assumed that the truck distribution will be the same in the case of no congestion. For the passenger car distributions this will not be the case. The passenger car distributions on a carriageway that consists of two traffic lanes (no congestion) is 70% on the right-hand lane and 30% on the left-hand lane. In case of congestion this distribution shifts to 30% on the right-hand lane and 70% on the left-hand lane. For reasons of simplification, the final model will use the average value for the right-hand lane with and without congestion. This corresponds to a passenger car distribution of 50% (70%+30% / 2) for both traffic lanes. So it is assumed that on both traffic lanes the same number of passenger cars will drive.

For a carriageway that consists of three traffic lanes, the truck distributions is taken as follows: right-hand lane 90%, centre lane 10% and left-hand lane 0% (see figure 6-36). In case of congestion it is assumed that this will also be the case. For the passenger car distributions this is 60% on the right-hand lane, 35% on the centre lane and 5% on the left-hand lane. In case of congestion, these values become 15% on the right-hand lane, 50% on the centre lane and 35% on the left-hand lane. The average value for the right lane (with and without congestion) is 37.5% (60%+15% / 2).

The skid resistance tests in the Netherlands and A70 (Bamberg) were carried out on the right-hand lane. So the distribution values of the right lane will be used to correlate the decrease of skid resistance with traffic.

The calculation of the number of axle loads is demonstrated for three cases.

A1 Apeldoorn:

AADT:	32000 vehicles per direction
Freight traffic:	15 %
Car traffic:	85 %
Traffic lanes (per direction):	2

Calculation of the number of axle loads per year for the right-hand lane:
32000*0.15*0.95*3.5 + 32000*0.85*0.5*2 = 15960 + 27200 = 43160 axle loads

Calculation of decrease of SR per axle loads:

Greywacke:	-0.0143 / 43160 = - $3.31 \cdot 10^{-7}$
Dutch gravel:	-0.0209 / 43160 = - $4.84 \cdot 10^{-7}$
Porphyry:	-0.0175 / 43160 = - $4.05 \cdot 10^{-7}$

Mini model 1 (Greywacke 60, PAC):	SR = 0.61 - $3.31 \cdot 10^{-7}$ * axle loads
Mini model 2 (Dutch gravel 54, PAC):	SR = 0.58 - $4.84 \cdot 10^{-7}$ * axle loads
Mini model 3 (Porphyry 52, PAC):	SR = 0.51 - $4.05 \cdot 10^{-7}$ * axle loads

A70 Bamberg Germany:

AADT:	35000 vehicles per direction
Freight traffic:	15 %
Car traffic:	85 %
Traffic lanes (per direction):	2

Calculation of the number of axle loads per year for the right-hand lane:
35000*0.15*0.95*3.5 + 35000*0.85*0.5*2 = 17456 + 29750 = 47206 axle loads

Calculation of decrease of SR per axle loads:

Greywacke:	-0.0024 / 47206 = - $5.08 \cdot 10^{-8}$
Moraine-1:	-0.003 / 47206 = - $6.36 \cdot 10^{-8}$
Diabase:	-0.0104 / 47206 = - $2.20 \cdot 10^{-7}$
Basalt:	-0.013 / 47206 = - $2.75 \cdot 10^{-7}$
Moraine-2:	-0.0102 / 47206 = - $2.16 \cdot 10^{-7}$
Dolomite:	-0.018 / 47206 = - $3.81 \cdot 10^{-7}$
Basalt:	-0.0046 / 47206 = - $9.78 \cdot 10^{-8}$

Mini model 5 (Greywacke 59, SMA):	SR = 0.52 – $5.08 \cdot 10^{-8}$ * axle loads
Mini model 7 (Moraine-1 53, SMA):	SR = 0.54 – $6.36 \cdot 10^{-8}$ * axle loads
Mini model 8 (Moraine-2 44, SMA):	SR = 0.46 – $2.16 \cdot 10^{-7}$ * axle loads
Mini model 9 (Basalt 47, SMA):	SR = 0.48 – $2.75 \cdot 10^{-7}$ * axle loads
Mini model 10 (Diabase 55, SMA):	SR = 0.51 – $2.20 \cdot 10^{-7}$ * axle loads
Mini model 11 (Dolomite 40, SMA):	SR = 0.44 – $3.81 \cdot 10^{-7}$ * axle loads
Mini model 13 (Basalt 47, DAC):	SR = 0.47 – $9.74 \cdot 10^{-8}$ * axle loads

N732 Lonneker-Losser:

AADT:	6700 vehicles per direction
Freight traffic:	7 %
Car traffic:	93 %
Traffic lane (per direction):	1

Calculation of the number of axle loads per year for the right-hand lane:
6700*0.93*2.0 + 6700*0.07*3.5 = 12462 + 1642 = 14104 axle loads

Calculation of decrease of SR per axle loads:
Moraine: -0.0369 / 14104 = $2.61 \cdot 10^{-6}$
Greywacke: -0.0208 / 14104 = $1.47 \cdot 10^{-6}$
Moraine: -0.0183 / 14104 = $1.30 \cdot 10^{-6}$

Mini model 4 (Moraine 53, PAC):	SR = 0.54 – $2.61 \cdot 10^{-6}$ * axle loads
Mini model 6 (Greywacke 61, SMA):	SR = 0.57 – $1.47 \cdot 10^{-6}$ * axle loads
Mini model 12 (Moraine 53, DAC):	SR = 0.52 – $1.30 \cdot 10^{-6}$ * axle loads

Decrease of friction coefficient (SR) per axle load vs. corresponding PSV
In this part of this study, the calculated decrease of the friction coefficient will be plotted against the corresponding PSV of the mineral aggregates to check if there is a correlation between these two parameters. The result of this analysis can be seen in Appendix B, figure B-1. From the result, it can seen that the decrease of the friction coefficient per axle load does not correlate with the PSV of the mineral aggregate (the data points are scattered and do not lie on a line). It was expected that these two parameters correlate with each other (the greater the PSV, the slower the friction coefficient decreases due to axle loads). A possible explanation for this could be that the assumed traffic distribution is not correct.

6.6 Proportional Distribution of the Total Expected Crash Rate

In this section, the proportional distribution of the total expected crash rate due to the road surface properties will be determined. This will be done because such a distribution was not found during the literature study. In order to come up to a proportional distribution, a method was invented to determine the proportional distribution. The steps that have been taken in order to obtain the proportional distribution will now be described extensively.
In the first step, the minimum and maximum values for the road surface properties are listed. These values can be seen in the second and third column of table 6-14. As maximum value for the skid resistance a friction coefficient value of 0.6 is assumed. This is done due to the fact that almost all the evaluated mineral aggregates in this study have an initial friction coefficient lower than this value (with the exception of the mineral aggregate greywacke in PAC). As minimum value for the road surface property roughness an IRI-value of 1.0 m/km is assumed. This is done due to the fact that it is almost impossible to obtain an IRI-value lower than 1.0 m/km when constructing a new road. In the second step, the crash rates are determined for the previously mentioned minimum and maximum values (third and fourth column). The crash rates are determined by using equations 21 & 23 and tables 5-5 & 5-6. In the third step, the average crash rate is determined for each road surface property (fifth column). Finally, the proportional distribution is determined. To illustrate how this is done an example is given for the road surface property skid resistance: 909.15/991.95= 92%. The percentages of the total expected crash rate in which the road surface properties are responsible for, are given in the last column of table 6-14. Figure 6-37 (pie chart) presents the calculated proportional distribution. From this figure it can be clearly seen that the skid resistance appears to be by far the most important property of the road surface with regard to the traffic safety. The road surface property crossfall does not form part of this method that is used to determine the proportional distribution.

Table 6-14: The proportional distribution of the total expected crash rate due to the road surface properties.

Road surface property	Minimum	Maximum	Crash rate [acc. per 100 mio.vhc.km]	Crash rate [acc. per 100 mio.vhc.km]	Average crash rate [acc. per 100 mio.vhc.km]	Percentage [%]
Skid resistance [friction coefficient]	0.38	0.60	272.3	1546.0	909.15	92.0
IRI	1.0 m/km	3.5 m/km	19.0	40.0	29.5	3.0
Texture	0.4 mm	1.9 mm	28.8	8.8	18.8	2.0
Rutting	0.0 mm	18.0 mm	37	32	34.5	3.0
				Totaal:	**991.95**	**100 %**

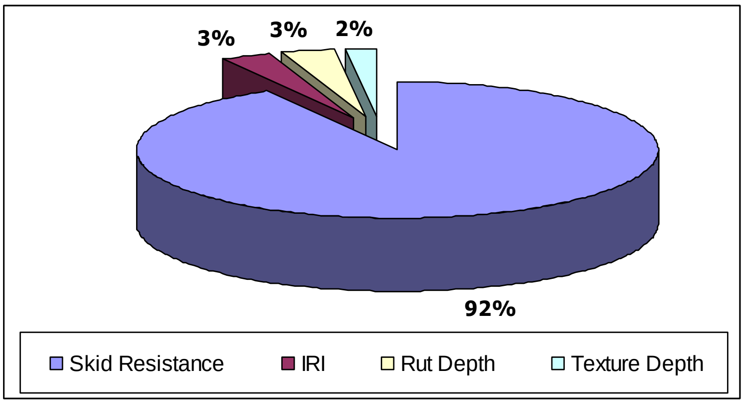


Figure 6-37: Proportional distribution of the total expected crash rate due to the road surface properties.

6.7 Cost-Benefit Analysis

In this part of this study, a cost-benefit analysis (CBA) will be performed to determine which mineral aggregate is the most sustainable one (from an economic perspective). This will be done by weighing the maintenance cost to improve the skid resistance and the construction cost against the savings on fatal accidents.

- **Maintenance Cost to Improve Skid Resistance**

The cost of a wearing course consists of three components. These three components are the construction cost, small maintenance cost and large maintenance cost. Cleaning of PAC pavement is an example of small maintenance. Replacement of the wearing course belongs to large maintenance cost. The type of material is not the only factor which determines the costs of a wearing course but also the type of maintenance that should be carried out.

The maintenance cost to improve the skid resistance within a certain period depends on mainly three factors. These are the chosen maintenance measure, the costs corresponding to that maintenance measure and the number of times that maintenance is required. In case of insufficient friction on PAC wearing courses, replacement of the wearing course is the common maintenance measure. If a shorter service life is acceptable, only the upper part (25 mm) of the PAC needs to be replaced with a new thin layer of PAC. Alternative maintenance measures are roughening of the polished stones by means of rays (with e.g. sand and crushed stone) or by bringing small scratches to the stones of the PAC (by rotating discs with diamonds). However, it must be said that the last two mentioned alternatives have normally a relatively short term effect. This means that these alternatives are not cost effective.

Replacements of PAC wearing courses in order to improve the skid resistance are most of the time performed over the width of only one traffic lane. The maintenance cost for this amounts €90,000 per km [Groenendijk 2013] for one traffic lane. This amount does not include the milling of the old wearing course (cost of milling amounts approximately €19,000 per km, see appendix C). The cost for a thin layer PAC amounts €60,000 per km. Replacement of SMA and DAC wearing courses (large maintenance) in order to improve the skid resistance amounts approximately €40,000 per km for SMA and €60,000 for DAC per km. Beside these maintenance costs, one must not forget the costs of traffic measures which are also a whole lot of money.

- **Benefit: Saving on Fatal Accidents due to Improved Skid Resistance**

In the Netherlands, for the evaluation of the traffic safety on motorways Rijkswaterstaat (RWS) uses a method where only fatal accidents and accidents with serious injuries are accounted. The costs corresponding to a victim due to a fatal accident amount approximately €2.6 million [SWOV,2012]. The costs corresponding to a serious injury in an accident amount €620.000. These amounts of money include the medical cost of injured victims, production loss of victims, loss of quality of life of injured victims, material costs, handling costs, traffic delay costs and traffic measure costs due to the accident. It seems that the costs for accidents with no serious injuries and Material Damage Only (MDO) are not taken into consideration but these are already settled in the costs of an accident with a serious injury.

Results of Calculations Based on an Example

With the use of the previously mentioned values, a cost-benefit analysis will be performed as an example to illustrate how the problem stated in subsection 1.2 (figure 1-4) can be solved. It must be said that this is a method to determine which type of wearing course is the most sustainable one. Beware that the calculated values are an approximation and not an exact value.

Figure 6-38 illustrates the expectation of the accidents costs during a period of 20 years and the cost due to maintenance. In this example, maintenance should be performed after a service life of 16 years. The ideal situation in this case will be if the accident cost (without maintenance) is higher than the accident cost (with maintenance) plus the cost due to the maintenance. If this is not the case, it would be not necessary to perform maintenance to the road surface. By carrying such a cost benefit analysis for two types of wearing course, the most sustainable (economical) type of wearing course with regard to the traffic safety and maintenance costs can be determined. The wearing course with the lowest total cost (accidents cost + maintenance cost) during a period of 20 years is the most sustainable/economical one.

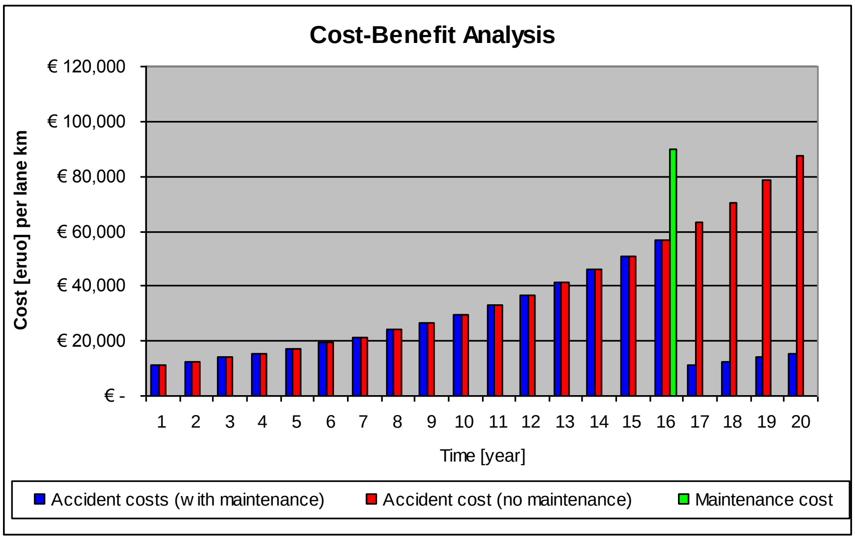


Figure 6-38: Example of cost-benefit analysis, greywacke in PAC.

7. DEVELOPED MODEL

In the previous chapter, the results of the performed data analysis were presented. These results are used for development of the final model. In the first part of this chapter the developed model will be illustrated and discussed. In the end of this chapter the interpretation of the results calculated by this model will be discussed.

7.1 Final Model

The final model is developed in Microsoft Excel. The final model consists of 6 worksheets. The first worksheet of the developed model is the opening window. This window shows a picture of an asphalt pavement with the five road surface properties that were investigated in this study (see figure 7-1).

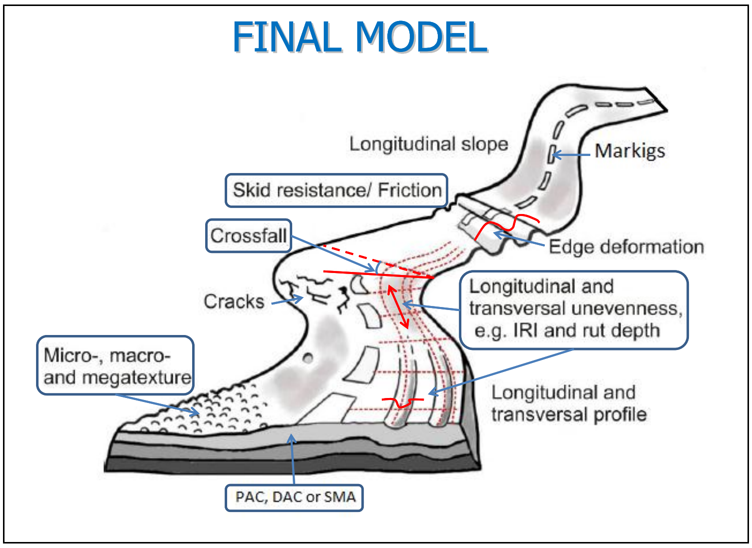


Figure 7-1: Opening window of the 'final model'.

The second worksheet is the most important one. In this worksheet the input data must be entered (see figure 7-2). The blue marked cells (column 3 of figure 7-2) are the only data that should be entered. The black ones change automatically by entering data in the blue cells. A drop down list for the blue cells is created to limit entries to certain items that were in advance defined making data entry easier in Excel. The data that should be entered by drop down list are the following: the type of wearing course, the type of mineral aggregate, asphalt thickness and the type of subgrade.
For the input data on traffic, these must be entered manually (no drop list). The data that should be entered regarding the traffic are the following: traffic intensity (AADT), growth rate, percentage truck traffic, number of traffic lanes (in one direction) and the speed limit. By choosing the number of traffic lanes the distribution of passenger cars and trucks will be automatically taken care of. These values are valid for the right-hand lane. The speed limit should be entered in order to calculate the correction factor that is needed to determine the reduced friction coefficient. Furthermore, the acceptance level for the road surface properties are given in the second worksheet. After entering the input data of the designed asphalt pavement the graphical results are shown automatically together with the input data. The graphical results consist of two parts. In the first part the development of the road surface properties due to the entered traffic data are shown (see figure 7-3). In the second part the graphical result of the calculated crash rate, fuel consumption and traffic noise are shown. Examples can be seen in figure 7-4.
The red lines shown in the graphs presented in figure 7-3 indicate the acceptance level of the road surface properties. The two red lines shown in the texture-graph indicate the texture range (0.4 mm to 1.9 mm).

PAVEMENT INPUT		
Skid resistance	fric. coefficient [-]	-
	Initial fric. Coeefficient	0.61
	fric. decrease per year	-3.310E-07
	wearing course	PAC
	type of mineral aggregate	Greywacke 60
Roughness	IRI [m/km]	-
	Initial IRI [m/km]	1.00
	IRI increase per year	0.0840
	asphalt thickness: h [mm]	thin: h ≤ 200 mm
	subgrade	Clay
Rutting	rut depth [mm]	-
	wearing coarse	PAC
	a-value	1.907
	b-value	0.682
Crossfall	crossfall [%]	2.5
Texture	Mean Profile Depth [mm]	-
	initial texture depth [mm]	1.7
	delta	-6.55E-07
TRAFFIC INPUT		
		-
Traffic	intensity [AADT]	32000
	traffic lanes	2
	number of passed axles	43160
	percentage truck traffic	15
	growth rate [%]	0
	design life [year]	20
	[%] passenger car distribution (right lane)	50
	[%] truck distribution (right lane)	95
	speed limit	80
ACCEPTANCE LEVEL	fric. coefficient [-]	> 0,38
	IRI [m/km]	< 3,5
	rut depth [mm]	< 18
	crossfall [%]	1,0 < % < 5
	MPD - range [mm]	0,4 < mm < 1,9
WATER FILM	Rain intensity [mm/hr]	50
	Width carriageway [m]	12
	Crossfall	2.5
	ETD	1.56
	Water Film Depth [mm]	0.59
	CFWpredicted	0.87

Figure 7-2: Input data worksheet of the final model.

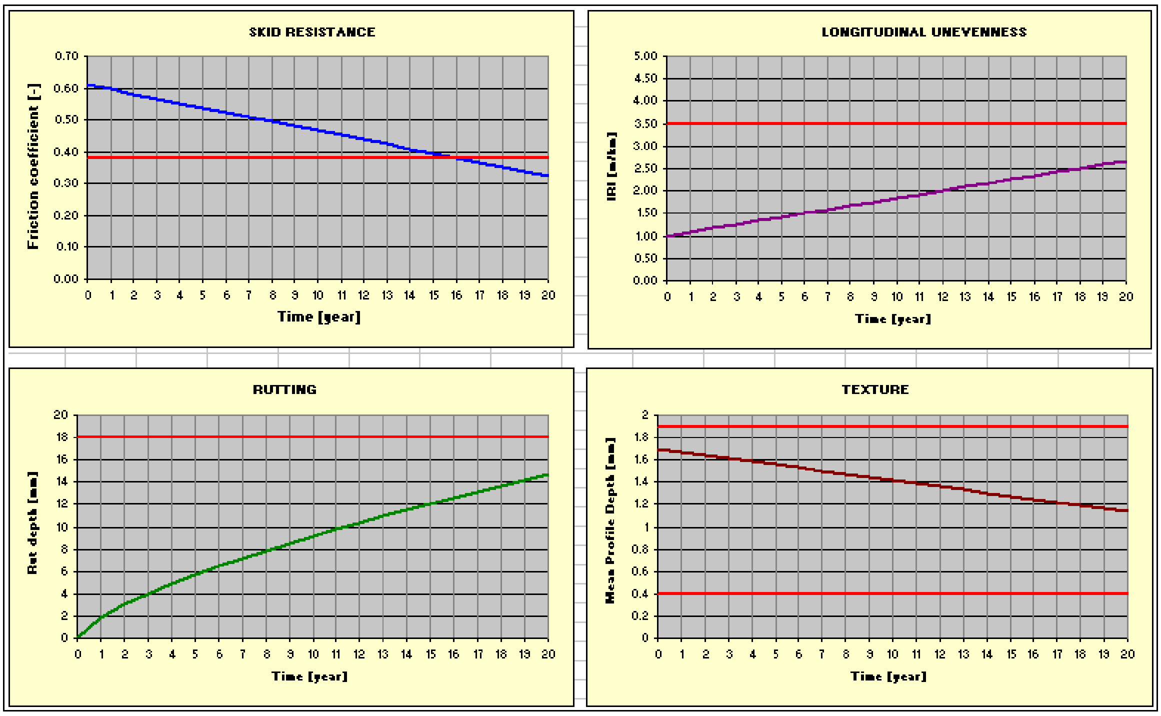


Figure 7-3: Graphical results of the expected developments of the road surface properties.

The first two bar charts in figure 7-4 show how the crash rate increases or decreases as the performance of the road surface properties also either decreases or increases. The third bar chart shows the total expected crash rate of the design asphalt pavement. This graph also shows the proportional distribution of the total expected crash rate due to the skid resistance, IRI, rut depth and the texture. The bottom three graphs show the expected change of traffic noise and fuel consumption during the design life of the asphalt pavement. Due to the fact that the part (%) of the total amount of fuel consumption which is consumed due to the texture and the IRI are not known, the final total fuel consumption cannot be indicated in one graph. The amount of consumed fuel due to the two indicative road surface properties (texture and IRI) cannot be summed up. This is the reason why two separately graphs are presented for the fuel consumption.

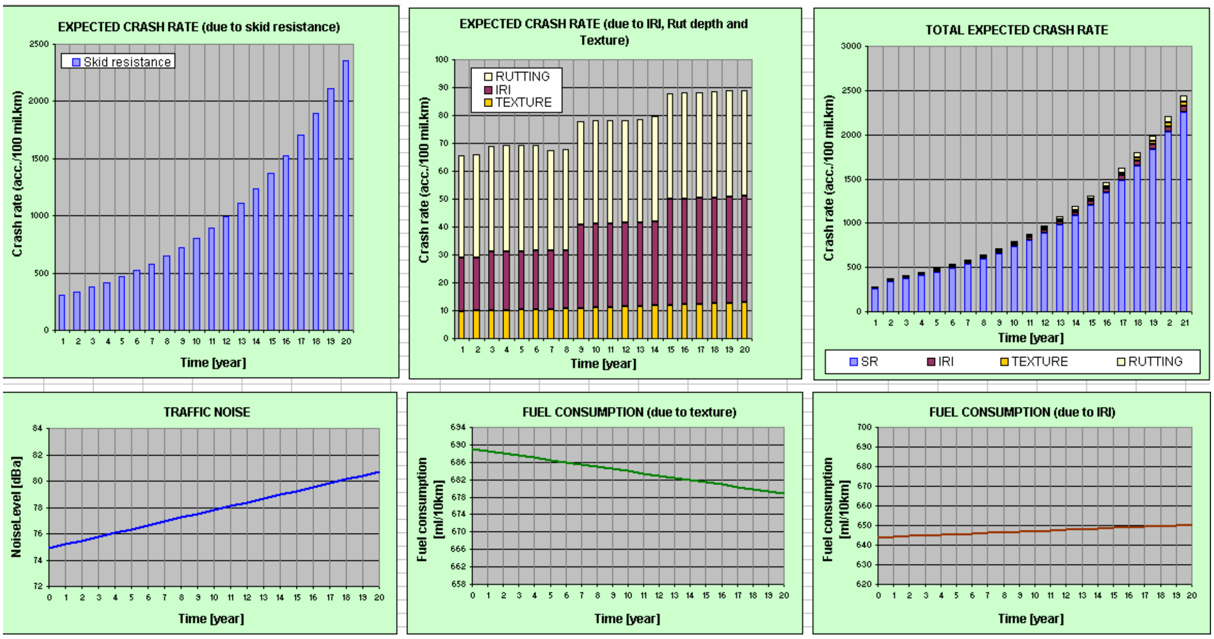


Figure 7-4: Graphical results of the expected crash rate, fuel consumption and traffic noise.

The third worksheet gives the numerical results belonging to the graphs shown in figure 7-3 and 7.4. The numerical results presented in the third worksheet can be seen in Appendix D, figure D-1. In the last three worksheets the matrices, equations and other calculated values are inserted. These data are obtained from the data analyses discussed in chapter 6 and the relevant information found in the literature study describing the relationships between the road surface properties and the environmental aspects traffic safety, fuel consumption and traffic noise.

7.2 Interpretation of the results

The results obtained from the final model must be interpreted as an indication of what can be expected from a designed asphalt pavement in terms of friction life, crash rate, fuel consumption and traffic noise. The results with respect to the expected crash rate can be used later for calculating the expected number of crashes. For this calculation, the equations 27 and 28 discussed in section 5.4 can be used. The results of the estimated fuel consumption and the traffic noise are relevant for road engineers to see what effect improvement of roughness and texture depth has on these environmental aspects. Furthermore, by changing one of the entries such as the type of wearing course, mineral aggregate or type of subgrade of the designed asphalt pavement, the effect on the development of the road surface properties and the expected crash rate, fuel consumption and traffic safety become immediately visible. This makes it easier for the road engineers to choose which designed asphalt pavement has the best result. The one with the best result can be considered as the most sustainable one.

7.3 Results of the Model based on an Example

This section shows the use fof the final model for a case study with two asphalt pavements with two different wearing courses and two different types of aggregate. The case study is typical for the situation in which a contractor may be if he wants to choose the best solution in a tender.

In this particular case, the contractor wants to compare the results in terms of traffic safety between a asphalt road with a DAC wearing course and one with a SMA wearing course. For the DAC wearing course the contractor has the mineral aggregate moraine in stock. This mineral aggregate has a PSV value of 53. For the SMA wearing course, the contractor wants to apply the mineral aggregate diabase with a PSV value of 55.

The choice of wearing course and mineral aggregate has no effect on the structural design of the pavement structure. In all cases thicknesses of all layers will be identical. For the case under study, the road is planned to be constructed on a sand subgrade. The road will consist of two traffic lanes, one in each direction of travel. The traffic intensity on the roads is 8200 vehicles per day (AADT). Furthermore, the truck traffic percentage is estimated to be 10%. More information about the pavement structure and traffic data for the two cases are presented below.

Pavement input:

Wearing course:	**SMA**	**DAC**
Mineral aggregate:	**Diabase**	**Moraine**
Crossfall	2.5%	
Subgrade:	Sand	
Road base:	Hydraulic mixed granulate	
Road base thickness:	300 mm	

Traffic input:

AADT:	8200 per direction
Traffic speed:	80 km/h
Traffic lane(s):	1 per direction
Truck traffic:	10%
Growth rate:	1%
Design life:	20 years

Use of the CROW pavement design software KMW 1.1 results into the following design:

- 200 mm asphalt
- 300 mm hydraulic mixed granulate
- 500 mm sand sub-base
- sand subgrade

The results of the deterioration of the four road surface properties are presented in figure 7-5. The orange-brown line corresponds to the SMA wearing course with diabase and the dark-blue line to the DAC wearing course with moraine as mineral aggregate.

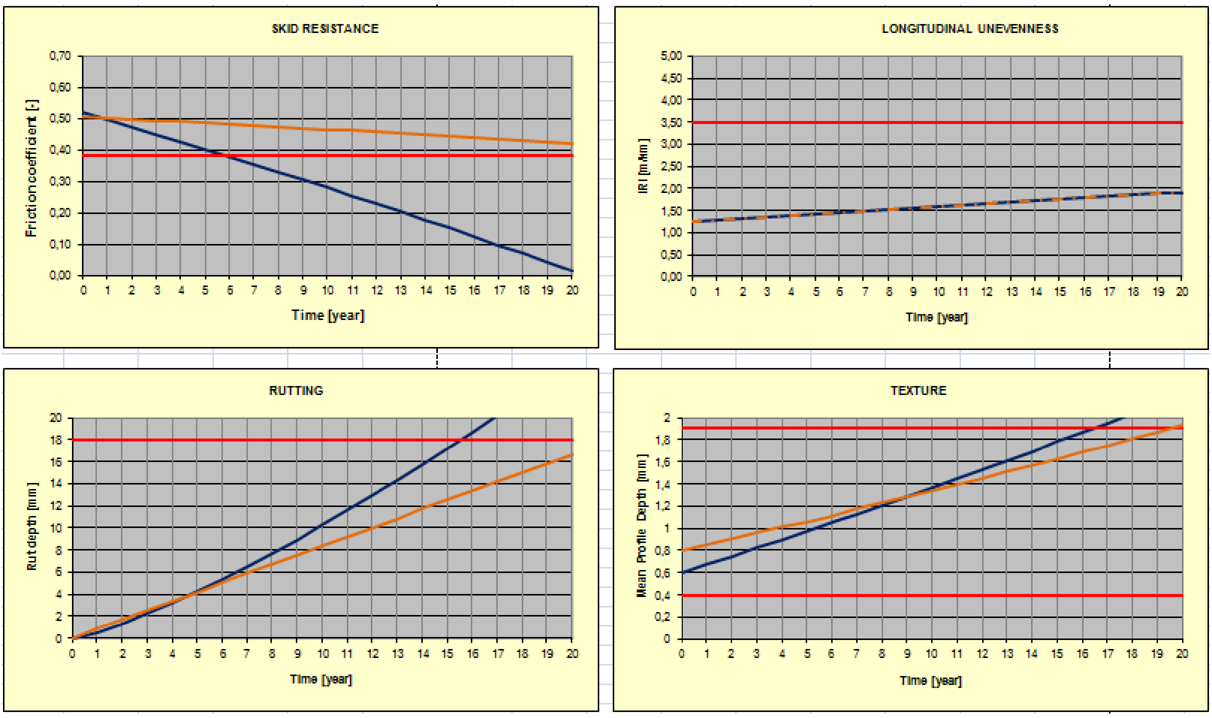


Figure 7-5: Results of the final model for two wearing courses and two types of aggregate (diabase & moraine).

The top-left graph shows that the mineral aggregate diabase provides the best performance over a period of 20 years. Application of this mineral aggregate leads to a a service life of just over 20 years. For the mineral aggregate moraine a friction service life of only 6 years is expected. This means that for this type of aggregate three times maintenance is required within a period of 20 years.
The development of roughness for both asphalt pavement is the same. This is due to the fact that the asphalt thickness and the subgrade for both asphalt pavement are identical (top right graph).

The bottom left graph shows the development of rutting. From this graph can be concluded that the SMA wearing course provides the best resistance to rutting compared to the DAC wearing course. For the DAC wearing course it takes approximately 15 years to reach the limit value of 18 mm. For the SMA wearing course, this is 21 years. The development of the texture depth can be seen in the bottom-right graph (figure 7-5).

The results regarding the expectations of the two designed asphalt pavements in terms of traffic noise, fuel consumption and crash rate are presented in figure 7-6.

The top most left graph shows that there is not much distinction between the performance of the SMA wearing course (orange) and the DAC wearing course (blue). The DAC pavement is more quiet at the start but gets noisier at the end. A similar pattern can be seen for the fuel consumption due to texture (see centre graph at the top). Since the IRI development is predominantly dependent on the stiffness of the subgrade, no distinction in fuel consumption due to development of IRI can be observed.

The bottom graphs present the crash rate as function of IRI, rutting and texture. The crash rate due to IRI increases with a jump after 15 years. This jump is due to the way of modelling. The crash rate versus IRI model uses classes of IRI-value as predictor and not a gradual relationship.
The expected crash rates due to changes of rutting and texture decrease within a period of 20 years. The crash rate of the SMA wearing course (orange), averaged over a period of 20 years appears to be marginally better rate than that of the DAC wearing course. This difference may be neglected.

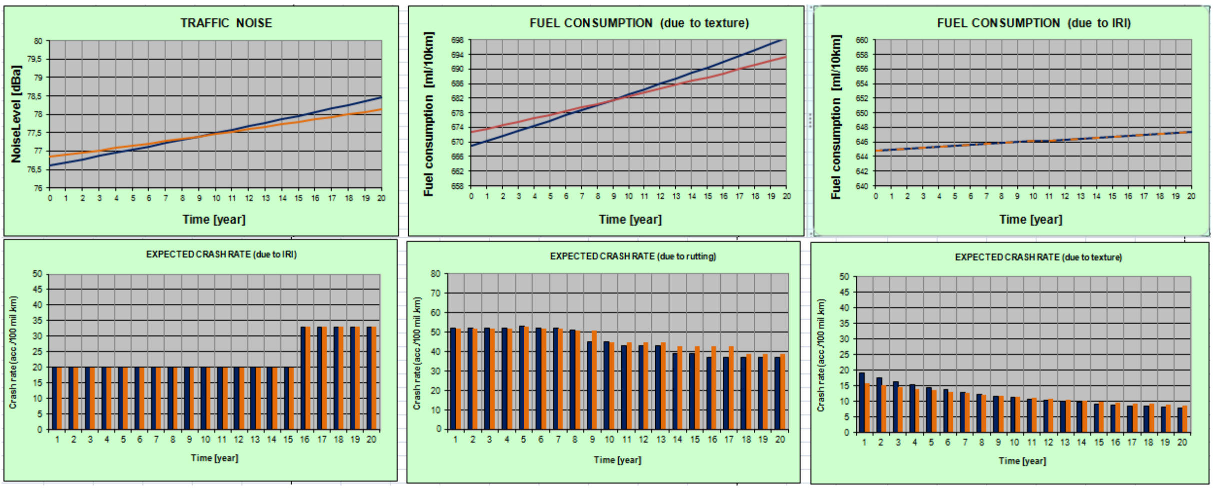


Figure 7-6: Results regarding the expectations from the two designed asphalt pavements.

The results of the expected crash rate due to declining skid resistance for the two cases can be observed in the figures 7-7 and 7-8. The mineral aggregate diabase in SMA offers a higher average skid resistance (0.464) during 20 years and results consequently into a lower average crash rate (819 acc. per 100 mio.vhc.km). The mineral aggregate moraine in DAC leads to an average skid resistance of 0.446 during 20 years, subsequently resulting into an average crash rate of 843 acc. per 100 mio.vhc.km.

This seems to point at SMA with diabase as the most feasible solution. However, the DAC wearing course (moraine) requires three times maintenance within a period of 20 years. This implies that there are years (the years immediately after maintenance) where the skid resistance is better than that of the SMA wearing course and years where skid resistance is just above the critical level (years prior to maintenance).

Figure 7-9 illustrates the expected crash rate due to the skid resistance for the two types of aggregate in one graph. On average there is not much difference between the two types of wearing courses and their mineral aggregates. The DAC wearing course with moraine has as enormous drawback, that three times maintenance needs to be applied. Determination of the aggregate costs for maintenance and costs associated to car crashes provides a more accurate decision for one of the two candidate wearing courses. Construction and milling costs can be found in Appendix C, these are given in order to perform further calculation if wanted.

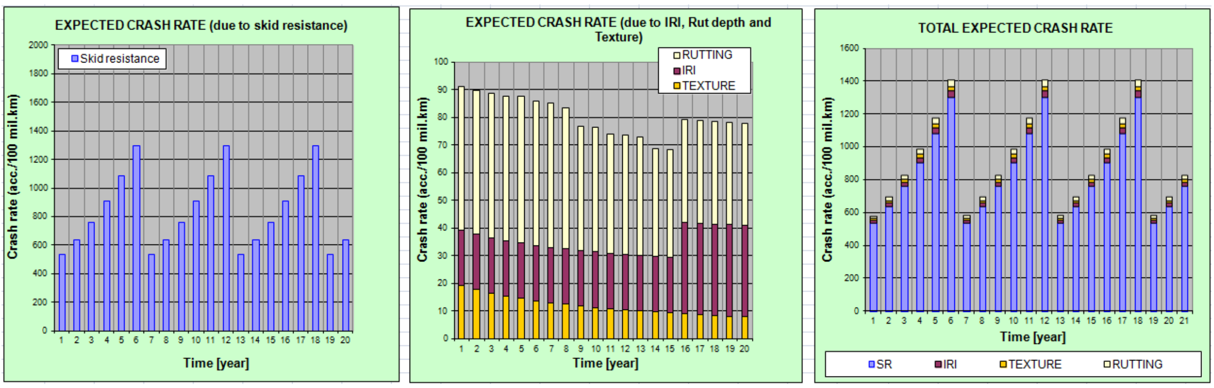


Figure 7-7: Expected crash rate for the designed asphalt pavement with a DAC wearing course (moraine).

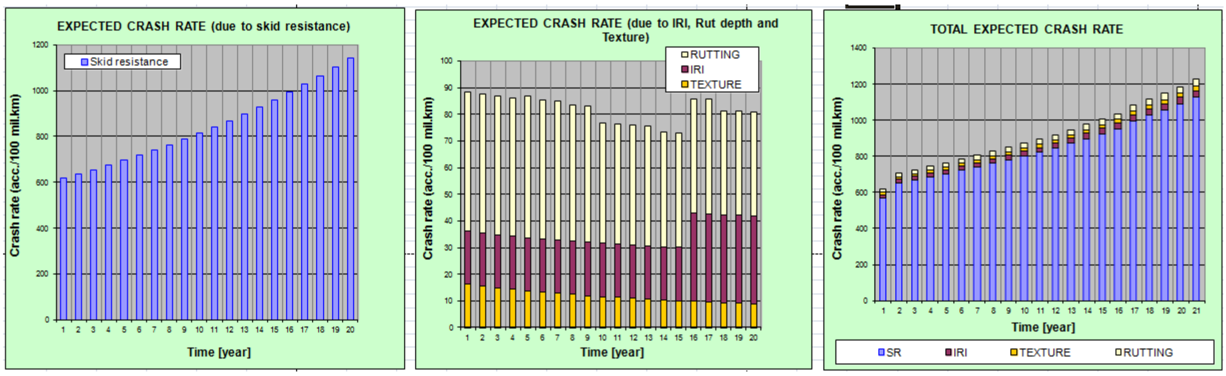


Figure 7-8: Expected crash rate for the designed asphalt pavement with a SMA wearing course (diabase).

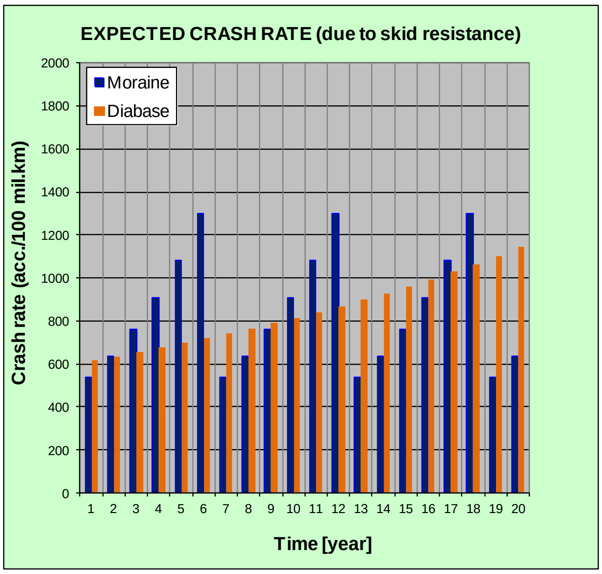


Figure 7-9: Expected crash rate due to the skid resistance for the two types of aggregate.

Ranking/Weighing of the three environmental aspects

In this part of this study, a possible method will be given to illustrate how the three environmental aspects could be ranked. The used method will not be explained extensively this is due to the fact that not much time was spent on this topic (weighing of the three environmental aspects).

For the ranking of the three environmental aspects table 7-1 will be used. The method which is used is based on the three principles of sustainable development (People, Planet, Prosperity). The idea behind this method is that there will be mainly looked at the positive effects that improvements of the evaluated environmental aspects have on the three principles of sustainable development. For example by improving the traffic safety the related social cost will be reduced. Due to the fact that this is a very important criteria, this criteria will be priced with five plus sign (maximum). By reducing the fuel consumption this will reduce the nuisance for the environment (four plus sign). The outcome after filling out the table completely is as following:

Table 7-1: Weighing of the three environmental aspects.

Environmental Aspects	Health/Social (people)	Environment (planet)	Economic (prosperity)	Score
Traffic safety	+++++	0	++++	9 → 38%
Fuel consumption	+	++++	+++	8 → 33%
Traffic noise	++++	++	+	7 → 29%

From table 7-1 it can be seen that the aspect 'traffic safety' the most important evaluated aspect is. The aspect fuel consumption takes the second place and the aspect traffic noise the last place.

7.4 Discussion

This section consists of two parts. The first part will touch on the obstacles (challenges) that have been encountered in this study. In the second part, the final model will be assessed by analysing the advantages and disadvantages of the model.

7.4.1 Challenges of the Research

This study started by reviewing literature for obtaining more knowledge on the background of sustainable development, traffic safety, fuel consumption and traffic noise. Many research reports were analysed to collect the necessary data on the relationships between road surface properties and the three environmental aspects traffic safety, fuel consumption and traffic noise. From all the studies used in this research, the following six appeared to be very useful for providing valuable data on the previously mentioned relationships. These six studies are:

- CROW (2005) – Traffic safety aspects of pavement surface properties (Original Dutch title: *Verkeersveiligheidsaspecten van wegoppervlakeigenschappen)*;
- VTI (2011) – Road users effect models;
- AUSTROADS (2013) – Development of Safety Related Investigatory Level Guidelines;
- Bendtsen (2000) – Rolling resistance, fuel consumption and emissions;
- Giezen et al. (2012) – Fuel consumption in relation to wearing course characteristics (Original Dutch title: *Brandstofverbruik in relatie tot wegdekkarakteristieken)*;
- Wayson (1998) – Relationship between pavement surface texture and highway traffic noise.

These studies are in more detail described in chapter three. This subsection will focus on the obstacles that were encountered during this research.
The various relationships between road surface properties and traffic safety found in the literature, showed significance differences in trends between the skid resistance (friction coefficient) and the crash rate. In many studies the unit used to express the crash rate could not be found or was not clearly presented. Only the name of these expressions was given (e.g. accident risk, accident ratio, accident number). In cases where information on the unit was presented, the units appeared to be different from study to study. No consistent unit was used for the expression of the crash rate. In some cases the crash rate was expressed as the number of accidents per million vehicle kilometres, but also units per 100 million vehicle kilometres were found.
In most cases it was not completely clear whether the crash rate contained only the accidents caused by the analyzed specific road surface property or that also accidents caused by other road surface imperfections were accounted for. Sometimes, it was not clear which type of accidents were accounted for in the given crash rate. Did the crash rate only contain the casualties or was the crash rate the total of all minor and major accidents in which traffic victims or injuries were involved.

Another problem that made this study even more complicated was the fact that the road surface properties do not have an independent, single impact on the traffic safety. In most cases pavement properties may be correlated. Crash rates resulting from analysis of a single pavement property may not be simply summed up to acquire the aggregate crash rate score. In one case, the crash rate may be the result of the skid resistance only while in another case it may be the result of a combination of two other road surface properties. So in an accident it is difficult to distinguish or to find confirmation on how many road surface properties were involved.

The methods used to measure the road surface properties are most of the time not the same from study to study. This is due to the fact that each country uses its own measurement and test method. This implies that many relationships between road surface properties and traffic safety found in the literature are often not directly applicable to the Dutch situation. The measurement data must first be converted to units or measurement methods that are normally used in the Netherlands.

Raveling, potholes or unevenness at e.g. bridges, but also sharp corners and bends may have their impact on the traffic safety. So the traffic safety is not only determined by the road surface properties alone but also by many other factors. These effects were not addressed in this study.

By reviewing all these mentioned points, the conclusion can be drawn that traffic safety is a very complex and sensitive subject to study. The factors that may have an impact on the traffic safety are numerous. It is also difficult to understand their influence on each other (interrelation). For this reason the results obtained from the developed model must be used with care and can only be seen as an indication of what can be expected in terms of environmental (sustainability) aspects when one or more pavement properties are varied. The results obtained from the model with respect to the traffic safety, fuel consumption and traffic noise may be different from reality. No validation study was performed to investigate the accuracy and precision of the model.

7.4.2 Assessment of the Final Model

This subsection touches on the advantages and disadvantages of the developed model. These pro's and con's are listed below.

Advantages of the model:

- The developed model can be used as a tool to help road engineers in designing sustainable roads (safer, greener and economical). Furthermore, the model helps road engineers in showing in advance what can be expected from variations in asphalt pavements and aggregates in terms of environmental and sustainability aspects.
- The model provides an indication on the development of the road surface properties with time or traffic. This allows for more accurate planning of future maintenance activities.
- The friction life of mineral aggregates can be evaluated by using the developed model.
- The model shows the difference between the expected noise level on three types of asphalt wearing course.

Disadvantages of the model:

- Some of the relationships used to develop the model are based on studies performed in other countries. The results are not always representative for or correlate to the Dutch situation.
- The model is based on the effect of individual road surface properties on the environmental aspects and not a combination of the road surface properties.
- An answer on the question "To what degree do road surface properties such as texture and roughness contribute to the total fuel consumption?" cannot be given by the model.
- Most of the mini models used in the final model are described with a linear equation. They could also be described with other types of equations that (perhaps) would provide for a better correlation with the performance of the mini models. Linear equations were used to keep the development of the model simple.
- The proportional distribution of the total expected crash rate due to the road surface properties used in this model is not supported by other studies. This is due to the fact that there are still no studies performed on this topic.

8. CONCLUSIONS AND RECOMMENDATIONS

This report presents the development of a model for the prediction of several environmental aspects for asphalt roads based on pavement structure and wearing course properties. The environmental aspects focus on traffic safety (crash rate), fuel consumption and traffic noise. The relationships between environmental aspect and pavement characteristic are retrieved from literature. The development or deterioration of pavement properties such as skid resistance, roughness, rutting, crossfall and texture are characterised on the basis of data collected on long-term pavement performance studies conducted by KOAC•NPC, SHRP-NL and others.

Road engineers can use the model as a tool to get an indication on how road (surface) properties will develop with time or traffic. Subsequently, the development of the road surface properties can be used to get an indication of what can be expected from the designed asphalt pavements in terms of crash rate, fuel consumption, traffic noise and surface characteristics. Simple-to-collect data on traffic and pavement data form the principal input of the model.
This chapter addresses the main conclusions and several recommendations for further research efforts.

8.1 Conclusions

The scope of this study was to develop a model that can be used by road engineers as a tool to get an indication of what can be expected from designed asphalt pavements and to obtain a better insight into the relationships between the road surface properties and the environmental aspects traffic safety, fuel consumption and traffic noise. The model should be able to indicate how the road surface characteristics develop with time or traffic. Subsequently, the model should be able to relate the progression of the road surface properties with the previously mentioned environmental aspects and finally to give reliable and usable results to the road engineer.
The conclusions of this study are split in three parts. First of all, conclusions will be given regarding the development of the road surface characteristics and on the performance of the evaluated mineral aggregates. Secondly, conclusions will be given concerning the analysis of the relationship between road surface properties and the traffic safety in terms of crash rate. At the end, conclusions will be given regarding the analysis for the effects of road surface properties on the fuel consumption and traffic noise.

Development of the road surface characteristics and performance of the mineral aggregates

- For many mineral aggregates reliable models could be developed for prediction of the change of the pavement surface property with time or number of axle passes.
- The Mean Profile Depth for porous asphalt wearing courses appears to decrease with time or traffic whereas it increases for dense asphaltic and stone mastic asphalt wearing courses. No clear explanation could be found for this phenomenon.
- From the evaluated mineral aggregates in this study, the mineral aggregate greywacke appears to have the best performance both in porous asphalt and stone mastic asphalt (SMA). The corresponding polishing rate is minimal and consequently a longer friction life and a lower average crash rate may be found in a period of 20 years. From the evaluated mineral aggregates in a SMA wearing course, the mineral aggregate dolomite results to have the worst performance.
- The calculated decrease of the friction coefficient due to the passing axle loads does not correlate with the polished stone value (PSV) of the mineral aggregates. It was expected that the higher the PSV would be, the more gradual the friction coefficient would decline with time and number of axle passes.

Analysis relationship between road surface properties and traffic safety

- Skid resistance appears to be by far the most prevailing pavement surface related factor on the value of the crash rate. The proportional contribution to the total crash rate is much more higher than for the other four road surface properties in the analysis (roughness, rutting, crossfall and texture depth). This means that the skid resistance has to be considered as the most important road surface property with regard to traffic safety. So it can be concluded that in order to increase the traffic safety it is wise to assure that the level of skid resistance remains high.
- The crash rate due to roughness has been found to increase considerably within a period of 20 years, while the Dutch major roads show that they are structural too stiff and do not exhibit significant roughness progression. Most of the roads in the Netherlands will not reach an IRI-value of 3.5 m/km within a period of 20 years. In other countries where higher IRI values are encountered, it is expected that the roughness will certainly have a huge impact on the crash rate.
- From the expected crash rate due to rutting it can be concluded that this crash rate either decreases or remains constant within a period of 20 years. So it can be concluded that the development of rutting has almost no negative effect on the crash rate with the exception of the problematic situation of standing water in the rut. In other words, the risk for aquaplaning during wet weather conditions can be considered as the only reason for ruts being a traffic accident risk. For the crash rate due to texture it can be estimated that this rate increases significantly (but only marginally when compared to skid resistance) when the texture depth decreases. Intersection and curves demand a higher level of skid resistance. This is due to the fact that the crash rate due to texture (MPD) on intersection and curves appears to be higher than on straight stretches of road (there exist a higher crash risk on curves and intersections).

Analysis relationship between road surface properties and the aspects fuel consumption and traffic noise:

- Due to the fact that the increase of roughness in The Netherlands is minimal and that the roughness is not allowed to exceed the intervention level of 3.5 m/km, it can be concluded that the development of roughness with time has hardly any influence on the crash rate and the fuel consumption. For other countries where much higher values for the roughness may be and will be found, it is expected that the roughness will have a much bigger impact on the crash rate and the fuel consumption. The increase of the fuel consumption due to an increase of the IRI from 1.0 m/km to 3.5 m/km is approximately 1.5%.
- The increase of the fuel consumption due to a decrease or an increase of the texture depth with 0.4 mm (MPD range of change) is approximately 1.1%. By looking at these values (1.1% and 1.5%) it can be concluded that overdue maintenance could lead to higher fuel consumption.
- The increase of traffic noise with time or traffic on a dense asphaltic wearing course (DAC) is much higher than that of a SMA wearing course. At first, no good indication could be found on how traffic noise would develop with time or traffic on a PAC wearing course. This was due to the fact that the MPD of porous asphalt wearing courses appears to decrease with time which resulted in predictions of decreasing traffic noise with time, which is not the case in reality. This problem was overcome by using an alternative modelling approach resulting in an annual increase of the traffic noise with 0.18 dB(A) for PAC wearing courses.

Final conclusion of this research programme

The study resulted in a tentative model that can be used by road engineers to design environment-friendly (sustainable) roads and also to get an indication of what can be expected from designed asphalt pavements in terms of crash rate, fuel consumption and traffic noise. Subsequently it can be concluded, that the main objective of this study is achieved.

8.2 Recommendations

As can be read in the previous chapters of this report, many studies were used to gather the necessary relationships between the road surface properties and the crash rate. A major part of these studies were not performed in the Netherlands but in other countries. This implies that the results obtained from the developed model may not be hundred percent representative for the Dutch situation. Because the Dutch roads are much safer in comparison with many other countries, the found crash rate predictions based on foreign studies may be on the high side and may need some fine-tuning to adopt them to the Dutch situation.

The increase of the MPD for dense graded wearing courses in this study may be explained by the development of raveling. Extra research efforts are needed to verify this hypothesis.
This requires that in addition to the MPD data of the investigated road sections, raveling data of these road sections must also be made available. Moreover, it is also recommended to study the effects that raveling has on the traffic safety (fright reactions due to flying stones dislodged from the pavement).

The final model is developed on the basis of data obtained for three types of wearing course. To extend the applicability of the model and to make it more valuable, further development of the final model is recommended. This can be organised by analysing data from other types of wearing course e.g. double layer porous asphalt (DLPA), asphalt emulsion concrete (AEC) and other surface types (e.g. concrete surfaces).

This research has focused on three environmental aspects. To make the final model and this research also more valuable, it is recommended to continue and extend this research by performing a Life Cycle Analysis (LCA) of the evaluated mineral aggregates in this research (sustainability aspects). This will provide more insight into the term of sustainability itself.

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APPENDIX – A

The method used to determine the road surface correction factor regarding the produced traffic noise is described in CROW Publication 316 [CROW, 2012]. The road surface correction factor indicate the effect of wearing courses on the produced traffic noise due to the passing vehicles.

Road Surface Correction:

The equations used to determine the road surface correction are the following:

$$C_{road\ surface,i,m}\ (v_m) = C_{initial,i,m}\ (v_m) + C_{time,i,m} \quad \rightarrow \quad C_{road\ surface,i,m}\ (v_m) = \sigma_m + \tau_m \log\ (v_m / v_{0,m})$$

Where:

$C_{initial,i,m}$ = initial correction factor [dB(A)]

$C_{time,i,m}$ = ageing correction [dB(A)] (the ageing correction $C_{time,i,m}$ is already processed in the value of σ_m.)

σ_m = parameter [-]

τ_m = parameter [-]

v_m = speed of traffic [km/h]

$v_{0,m}$ = reference speed [km/h]

$C_{road\ surface,i,m}$ = road surface correction [dB(A)]

Table A-1: Values for the parameters $\sigma_{i,m}$, σ_m and τ_m in order to calculate the road surface correction factor for light (m=1) vehicles (CROW Publication 316).

Lichte motorvoertuigen		Snelheidsbereik		SRMI	SRMII: σi,m								SRMI/SRMII
Nr	Wegdektype/-product	Vmin1	Vmax1	σm	63Hz	125Hz	250Hz	500Hz	1kHz	2kHz	4kHz	8kHz	τm
0	referentiewegdek	30	130	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	1L ZOAB	50	130	-1.4	0.5	3.3	2.4	3.2	-1.3	-3.5	-2.6	0.5	-6.5
2	2L ZOAB	50	130	-4.5	0.4	2.4	0.2	-3.1	-4.2	-6.3	-4.8	-2.0	-3.0
3	2L ZOAB fijn	80	130	-6.5	-1.0	1.7	-1.5	-5.3	-6.3	-8.5	-5.3	-2.4	-0.1
4a	SMA 0/5	40	80	-1.9	1.1	-1.0	0.2	1.3	-1.9	-2.8	-2.1	-1.4	-1.0
4b	SMA 0/8	40	80	-0.8	0.3	0.0	0.0	-0.1	-0.7	-1.3	-0.8	-0.8	-1.0
5	uitgeborsteld beton	70	120	1.9	1.1	-0.4	1.3	2.2	2.5	0.8	-0.2	-0.1	1.4
6	geoptim. uitgeborsteld beton	70	80	0.3	-0.2	-0.7	0.6	1.0	1.1	-1.5	-2.0	-1.8	1.0
7	fijngebezemd beton	70	120	2.0	1.1	-0.5	2.7	2.1	1.6	2.7	1.3	-0.4	7.7
8	oppervlakbewerking	50	130	2.9	1.1	1.0	2.6	4.0	4.0	0.1	-1.0	-0.8	-0.2
9a	elementenverharding keperverband	30	60	2.4	8.3	8.7	7.8	5.0	3.0	-0.7	0.8	1.8	2.5
9b	elementenverharding niet in keperv	30	60	6.1	12.3	11.9	9.7	7.1	7.1	2.8	4.7	4.5	2.9
10	stille elementenverharding	30	60	-2.0	7.8	6.3	5.2	2.8	-1.9	-6.0	-3.0	-0.1	-1.7
11	dunne deklagen A	40	130	-3.4	1.1	0.1	-0.7	-1.3	-3.1	-4.9	-3.5	-1.5	-2.5
12	dunne deklagen B	40	130	-5.0	0.4	-1.3	-1.3	-0.4	-5.0	-7.1	-4.9	-3.3	-1.5

Table A-2: Values for the parameters $\sigma_{i,m}$, σ_m and τ_m in order to calculate the road surface correction factor for middle and heavy (m=2 and m=3) vehicles (CROW Publication 316).

Zware motorvoertuigen		Snelheidsbereik		SRMI	SRMII: σi,m								SRMI/SRMII
Nr	Wegdektype/-product	Vmin1	Vmax1	σm	63Hz	125Hz	250Hz	500Hz	1kHz	2kHz	4kHz	8kHz	τm
0	referentiewegdek	30	100	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	1L ZOAB	70	100	-3.1	0.9	1.4	1.8	-0.4	-5.2	-4.6	-3.0	-1.4	0.2
2	2L ZOAB	70	100	-5.2	0.4	0.2	-0.7	-5.4	-6.3	-6.3	-4.7	-3.7	4.7
3	2L ZOAB fijn	70	100	-5.3	1.0	0.1	-1.8	-5.9	-6.1	-6.7	-4.8	-3.8	-0.8
4a	SMA 0/5	40	80	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4b	SMA 0/8	40	80	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	uitgeborsteld beton	70	100	-0.3	0.0	1.1	0.4	-0.3	-0.2	-0.7	-1.1	-1.0	4.4
6	geoptim. uitgeborsteld beton	70	80	-1.6	-0.3	1.0	-1.7	-1.2	-1.6	-2.4	-1.7	-1.7	-6.6
7	fijngebezemd beton	70	90	1.7	0.0	3.3	2.4	1.9	2.0	1.2	0.1	0.0	3.7
8	oppervlakbewerking	50	100	-0.5	0.0	2.0	1.8	1.0	-0.7	-2.1	-1.9	-1.7	1.7
9a	elementenverharding keperverband	30	60	3.5	8.3	8.7	7.8	5.0	3.0	-0.7	0.8	1.8	2.5
9b	elementenverharding niet in keperv	30	60	6.9	12.3	11.9	9.7	7.1	7.1	2.8	4.7	4.5	2.9
10	stille elementenverharding	30	60	1.4	0.2	0.7	0.7	1.1	1.8	1.2	1.1	0.2	0.0
11	dunne deklagen A	40	100	-1.3	1.6	1.3	0.9	-0.4	-1.8	-2.1	-0.7	-0.2	0.5
12	dunne deklagen B	40	100	-1.3	1.6	1.3	0.9	-0.4	-1.8	-2.1	-0.7	-0.2	0.5

Table A-3: Values for the parameters ΔL_m, $\Delta L_{i,m}$ and τ_m in order to calculate the initial correction factor for the three vehicle categories m=1, m=2 and m=3 (CROW Publication 316).

categorie	wegdektype		ΔL_m	ΔL_i_m								τ_m	vmin	vmax
				i=1	i=2	i=3	i=4	i=5	i=6	i=7	i=8			
				63	125	250	500	1000	2000	4000	8000			
1	ZOAB	m=1	-3.0	0.9	3.0	2.1	3.0	-4.0	-5.1	-4.1	-1.1	-6.5	50	130
		m=2,3	-4.5	1.2	1.2	1.4	-1.7	-7.2	-5.6	-4.2	-3.1	0.2	70	100
2	Tweelaags ZOAB	m=1	-6.3	0.3	1.9	-1.0	-4.9	-6.0	-7.8	-6.2	-3.2	-3.0	50	130
		m=2,3	-7.0	0.3	-0.6	-1.9	-6.8	-7.6	-7.1	-5.5	-4.5	4.7	70	100
3	Fijn tweelaags ZOAB	m=1	-8.4	-1.0	1.1	-2.7	-7.5	-8.1	-10.2	-7.0	-4.1	-0.1	80	130
		m=2,3	-6.7	1.5	-0.2	-2.4	-7.3	-6.9	-6.9	-4.9	-4.4	-0.8	70	100
4a	SMA-NL5	m=1	-2.9	0.0	-2.1	-0.9	0.2	-3.0	-3.9	-3.2	-2.5	-1.0	40	80
		m=2,3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	40	80
4b	SMA-NL8	m=1	-1.9	-0.8	-1.1	-1.1	-1.2	-1.8	-2.4	-1.9	-1.9	-1.0	40	80
		m=2,3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	40	80
5	Uitgeborsteld beton	m=1	0.8	0.0	-1.5	0.2	1.1	1.4	-0.3	-1.3	-1.2	1.4	70	120
		m=2,3	-0.4	0.0	1.1	0.4	-0.3	-0.2	-0.7	-1.1	-1.0	4.4	70	100
6	Geoptimaliseerd uitgeborsteld beton	m=1	-0.9	-1.3	-1.8	-0.5	-0.1	0.0	-2.6	-3.1	-2.9	1.0	70	80
		m=2,3	-1.8	-0.3	1.0	-1.7	-1.2	-1.6	-2.4	-1.7	-1.7	-6.6	70	80
7	Fijngebezemd beton	m=1	0.9	0.0	-1.6	1.6	1.0	0.5	1.6	0.2	-1.5	7.7	70	120
		m=2,3	1.7	0.0	3.3	2.4	1.9	2.0	1.2	0.1	0.0	3.7	70	90
8	Oppervlakbewerking	m=1	1.7	0.0	-0.1	1.5	2.9	2.9	-1.0	-2.1	-1.9	-0.2	50	130
		m=2,3	-0.9	0.0	2.0	1.8	1.0	-0.7	-2.1	-1.9	-1.7	1.7	50	100
9a	Elementenverharding in keperverband	m=1	0.8	8.1	8.0	7.1	3.9	1.2	-1.9	-0.3	1.6	2.5	30	60
		m=2,3	0.8	8.1	8.0	7.1	3.9	1.2	-1.9	-0.3	1.6	2.5	30	60
9b	Elementenverharding niet in keperverband	m=1	4.4	12.1	11.2	9.0	6.0	5.3	1.6	3.6	4.3	2.9	30	60
		m=2,3	4.4	12.1	11.2	9.0	6.0	5.3	1.6	3.6	4.3	2.9	30	60
10	Stille elementenverharding	m=1	-3.5	7.6	5.6	4.5	1.7	-3.7	-7.2	-4.1	-0.3	-1.7	30	60
		m=2,3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	30	60
11	Dunne deklagen A	m=1	-5.5	1.3	0.6	-1.1	-1.8	-5.2	-8.0	-5.3	-2.5	-2.5	40	130
		m=2,3	-3.1	2.9	2.2	1.4	-1.0	-3.8	-4.4	-1.7	-0.6	0.5	40	100
12	Dunne deklagen B	m=1	-6.7	0.6	-0.8	-1.7	-0.9	-7.1	-10.2	-6.7	-4.3	-1.5	40	130
		m=2,3	-3.1	2.9	2.2	1.4	-1.0	-3.8	-4.4	-1.7	-0.6	0.5	40	100

APPENDIX – B

SR: skid resistance
CR: crash rate

Table B-1: Level of skid resistance per year with estimated crash rate (A1 Apeldoorn).

Greywacke			Dutch stone chips			Porhyry		
YEAR	**SR**	**CR**	**YEAR**	**SR**	**CR**	**YEAR**	**SR**	**CR**
1	0.60	3.03	**1**	0.56	4.00	**1**	0.49	6.61
2	0.58	3.38	**2**	0.54	4.68	**2**	0.48	7.55
3	0.57	3.76	**3**	0.52	5.48	**3**	0.46	8.61
4	0.55	4.19	**4**	0.50	6.42	**4**	0.44	9.83
5	0.54	4.67	**5**	0.48	7.52	**5**	0.42	11.22
6	0.52	5.20	**6**	0.45	8.80	**6**	0.41	12.80
7	0.51	5.80	**7**	0.43	10.31	**7**	0.39	14.61
8	0.50	6.46	**8**	0.41	12.07	**7.6**	0.38	15.81
9	0.48	7.20	**9**	0.39	14.13	**1**	0.49	6.61
10	0.47	8.02	**9.4**	0.38	15.05	**2**	0.48	7.55
11	0.45	8.93	**1**	0.56	4.00	**3**	0.46	8.61
12	0.44	9.95	**2**	0.54	4.68	**4**	0.44	9.83
13	0.42	11.08	**3**	0.52	5.48	**5**	0.42	11.22
14	0.41	12.34	**4**	0.50	6.42	**6**	0.41	12.80
15	0.40	13.75	**5**	0.48	7.52	**7**	0.39	14.61
16	0.38	15.32	**6**	0.45	8.80	**7.6**	0.38	15.81
16.1	0.38	15.49	**7**	0.43	10.31	**1**	0.49	6.61
1	0.60	3.03	**8**	0.41	12.07	**2**	0.48	7.55
2	0.58	3.38	**9**	0.39	14.13	**3**	0.46	8.61
3	0.57	3.76	**9.4**	0.38	15.05	**4**	0.44	9.83
3.9	0.55	4.15	**1**	0.56	4.00	**4.8**	0.43	10.92
			1.2	0.55	4.13			
Average	**0.500**	**7.281**		**0.475**	**8.411**		**0.438**	**10.362**

Table B-2: Level of skid resistance per year with estimated crash rate (Bamberg Germany).

Greywacke			Moraine 2			Diabase		
YEAR	**SR**	**CR**	**YEAR**	**SR**	**CR**	**YEAR**	**SR**	**CR**
1	0.52	5.28	**1**	0.54	4.73	**1**	0.50	6.27
2	0.52	5.18	**2**	0.53	4.83	**2**	0.49	6.78
3	0.53	5.09	**3**	0.53	4.94	**3**	0.48	7.33
4	0.53	5.00	**4**	0.53	5.06	**4**	0.47	7.93
5	0.53	4.91	**5**	0.53	5.17	**5**	0.46	8.58
6	0.53	4.82	**6**	0.52	5.29	**6**	0.45	9.28
7	0.54	4.73	**7**	0.52	5.41	**7**	0.44	10.04
8	0.54	4.65	**8**	0.52	5.54	**8**	0.43	10.86
9	0.54	4.56	**9**	0.51	5.66	**9**	0.42	11.74
10	0.54	4.48	**10**	0.51	5.79	**10**	0.41	12.70
11	0.55	4.40	**11**	0.51	5.93	**11**	0.40	13.74
12	0.55	4.32	**12**	0.50	6.06	**12**	0.39	14.86
13	0.55	4.24	**13**	0.50	6.20	**1**	0.50	6.27
14	0.55	4.17	**14**	0.50	6.34	**2**	0.49	6.78
15	0.56	4.09	**15**	0.50	6.49	**3**	0.48	7.33
16	0.56	4.02	**16**	0.49	6.64	**4**	0.47	7.93
17	0.56	3.95	**17**	0.49	6.79	**5**	0.46	8.58
18	0.56	3.88	**18**	0.49	6.94	**6**	0.45	9.28
19	0.57	3.81	**19**	0.48	7.10	**7**	0.44	10.04
20	0.57	3.74	**20**	0.48	7.27	**8**	0.43	10.86
	0.545	**4.466**		**0.509**	**5.910**		**0.451**	**9.359**

Table B-3: Level of skid resistance per year with estimated crash rate (Bamberg Germany).

Basalt			Moraine 1			Dolomite		
YEAR	**SR**	**CR**	**YEAR**	**SR**	**CR**	**YEAR**	**SR**	**CR**
1	0.47	8.02	**1**	0.45	9.13	**1**	0.42	11.26
2	0.45	8.84	**2**	0.44	9.86	**2**	0.40	12.90
3	0.44	9.75	**3**	0.43	10.65	**3**	0.39	14.77
4	0.43	10.76	**4**	0.42	11.50	**3.5**	0.38	15.81
5	0.42	11.87	**5**	0.41	12.42	**1**	0.42	11.26
6	0.40	13.09	**6**	0.40	13.41	**2**	0.40	12.90
7	0.39	14.44	**7**	0.39	14.49	**3**	0.39	14.77
8	0.38	15.93	**1**	0.45	9.13	**3.5**	0.38	15.81
1	0.47	8.02	**2**	0.44	9.86	**1**	0.42	11.26
2	0.45	8.84	**3**	0.43	10.65	**2**	0.40	12.90
3	0.44	9.75	**4**	0.42	11.50	**3**	0.39	14.77
4	0.43	10.76	**5**	0.41	12.42	**3.5**	0.38	15.81
5	0.42	11.87	**6**	0.40	13.41	**1**	0.42	11.26
6	0.40	13.09	**7**	0.39	14.49	**2**	0.40	12.90
7	0.39	14.44	**1**	0.45	9.13	**3**	0.39	14.77
8	0.38	15.93	**2**	0.44	9.86	**3.5**	0.38	15.81
1	0.47	8.02	**3**	0.43	10.65	**1**	0.42	11.26
2	0.45	8.84	**4**	0.42	11.50	**2**	0.40	12.90
3	0.44	9.75	**5**	0.41	12.42	**3**	0.39	14.77
4	0.43	10.76	**6**	0.40	13.41	**3.5**	0.38	15.81
						1	0.42	11.26
						2	0.40	12.90
						2.5	0.40	13.80
	0.427	**11.140**		**0.421**	**11.493**		**0.397**	**13.686**

Table B-4: PSV against the decrease of friction coefficient per axle load.

Mineral aggregate	PSV	Decrease of friction coefficient per axle load
Greywacke	60	-3.31E-07
Porphyry	52	-4.05E-07
Dutch gravel	54	-4.84E-07
Greywacke	59	-5.08E-08
Moraine	53	-6.36E-08
Diabase	55	-2.20E-07
Basalt	47	-2.75E-07
Moraine	44	-2.16E-07
Dolomite	40	-3.81E-07
Basalt	47	-9.74E-08

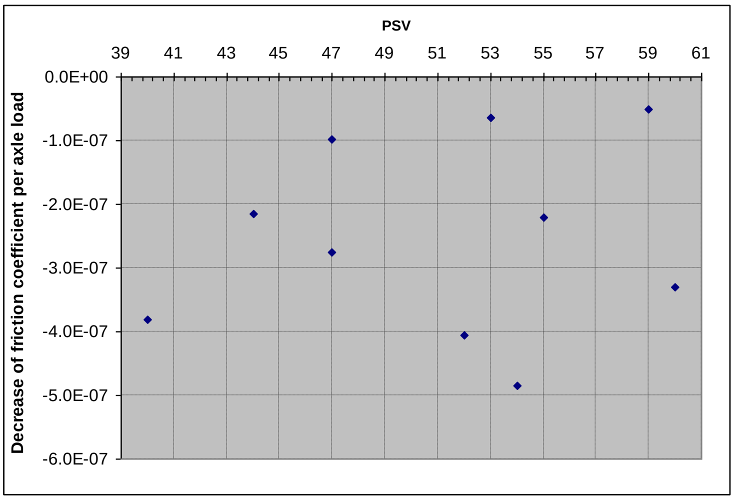

Figure B-1: PSV plotted against the decrease of friction coefficient per axle load.

Table B-5: Selected road sections for the modelling of IRI development.

	Road section	Annual IRI Increase	Subgrade	AADT	Asphalt Thickness
h ≤ 200 mm	1005	0,0345	Clay	2000	160
	1017	0,049	Clay	8000	*158*
	5042	0,0087	Clay	3600	126
	1041	0,029	Clay	10850	200
h > 200 mm	1015	0,028	Clay	2590	*272*
	1073	0,028	Clay	6115	291
	1079	0,052	Clay	6936	228
	1113	0,045	Clay + Sand	6131	451
	1126	0,049	Clay	14655	301
	5012	0,02	Clay	3250	229
	5026	0,0107	Clay	11500	242
	5036	0,0194	Clay	7500	314
h ≤ 200 mm	1026	0,0156	Sand	2200	140
	1121	0,0167	Sand	1050	145
	5001	0,0178	Sand	2625	102
	1002	0,04	Sand	900	143
	1004	0,0167	Sand	1925	195
	1012	0,015	Sand	1610	183
	5021	0,0183	Sand	5060	180
	1024	0,025	Sand	2900	180
	1038	0,0156	Sand	2700	162
	1043	0,0333	Sand	7380	168
h > 200 mm	1020	0,013	Sand	8455	212
	5003	0,0144	Sand	5450	204
	1018	0,052	Sand	5000	209
	1097	0,019	Sand	15600	235
	1078	0,021	Sand	6200	269
	1096	0,053	Sand	16000	322
	1101	0,045	Sand	17500	350
	1105	0,044	Sand	14500	456
	1106	0,032	Sand	10994	333
	1019	0,009	Sand	4275	286

APPENDIX - C

Calculations in order to describe the development of the MPD as a variable of the number of axle loads **.**

- N62 Westerscheldetunnelweg:

AADT:	8000 vehicles
Freight traffic:	15 %
Car traffic:	85 %
Traffic lanes (one direction):	2

- A8/A10 Coentunnel:

AADT:	50000 vehicles
Freight traffic:	15 %
Car traffic:	85 %
Traffic lanes (one direction):	2

Calculation:
8000*0.15*0.95*3.5 + 8000*0.85*0.5*2 = 3990 + 6800 = 10790 axle loads on the heaviest trafficked lane.

MPD (PAC): $- 0.0442 / 67438 = - 6.55 \cdot 10^{-7}$
MPD (SMA): $+0.0207 / 10790 = + 1.92 \cdot 10^{-6}$
MPD (DAC): $+0.0448 / 10790 = + 4.15 \cdot 10^{-6}$

Mini model 14 (PAC):	MPD = $1.7 - 6.55 \cdot 10^{-7}$ * axle loads
Mini model 15 (SMA):	MPD = $0.8 + 1.92 \cdot 10^{-6}$ * axle loads
Mini model 16 (DAC):	MPD = $0.6 + 4.15 \cdot 10^{-6}$ * axle loads

Calculation of the initial noise level ($dB_{initial}$) of PAC (page 50):

$dB_{initial}$ = 1.143 * 1.7 (average MPD of PAC) + 72.93 = *74.9 dB.*

Milling costs (costs per m^2)
- 25 mm: € 4,84
- 40 mm: € 4,91
- 65 mm: € 5,49
- 80 mm: € 9,36

Construction costs (laying of asphalt wearing course) (costs per m^2)
- 40 mm PAC 0/16: € 26,90
- 30 mm SMA 0/8: € 11,47
- 40 mm DAC 0/8: € 13,69
- 30 mm DAC 0/11: € 10,20
- 40 mm STAC 0/16: € 11,47
- 55 mm STAC 0/16: € 15,75
- 25 mm Thin layer (dunne deklaag): € 10.07

APPENDIX – D

PAVEMENT INPUT		Time [year]	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Skid resistance	fric. coefficient [-]	-	0.61	0.60	0.58	0.57	0.55	0.54	0.52	0.51	0.50	0.48	0.47	0.45	0.44	0.42	0.41	0.40	0.38	0.37	0.35	0.34	0.32
	reduced fric. coefficient due to the waterfilm		0.53	0.52	0.51	0.50	0.48	0.47	0.46	0.45	0.43	0.42	0.41	0.40	0.38	0.37	0.36	0.35	0.33	0.32	0.31	0.30	0.28
Roughness	IRI [m/km]	-	1.00	1.08	1.17	1.25	1.34	1.42	1.50	1.59	1.67	1.76	1.84	1.92	2.01	2.09	2.18	2.26	2.34	2.43	2.51	2.60	2.68
Rutting	rut depth [mm]	-	0.00	1.91	3.06	4.03	4.91	5.71	6.47	7.18	7.87	8.53	9.16	9.78	10.37	10.96	11.52	12.08	12.62	13.15	13.68	14.19	14.69
Crossfall	crossfall [%]	-	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
Texture	Mean Profile Depth [mm]	-	1.70	1.67	1.64	1.62	1.59	1.56	1.53	1.50	1.47	1.45	1.42	1.39	1.36	1.33	1.30	1.28	1.25	1.22	1.19	1.16	1.13
	Traffic Noise	-																					
	x-value	1.143																					
	y-value	72.93																					
TRAFFIC INPUT																							
Traffic	intensity [ADDT]	32000																					
	traffic lanes (one direction)	2																					
	number of passed axles	43160																					
	percentage truck traffic	15																					
	growth rate [%]	0																					
	design life [year]	20																					
	[%] passenger car distribution (right lane)	50																					
	[%] truck distribution (right lane)	95																					
		Time [year]	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
		Traffic [AADT]	0	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000	32000
		Cumulative [AADT]	0	32000	64000	96000	128000	160000	192000	224000	256000	288000	320000	352000	384000	416000	448000	480000	512000	544000	576000	608000	640000
		Passed axles	0	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160	43160
		Cum. passed axles	0	43160	86320	129480	172640	215800	258960	302120	345280	388440	431600	474760	517920	561080	604240	647400	690560	733720	776880	820040	863200
RESULTS	Crash Rate (acc./mio. veh.km)	**Due to SR**	272.3	303.3	337.9	376.3	419.2	466.9	520.1	579.4	645.3	718.8	800.7	891.9	993.5	1106.6	1232.7	1373.1	1529.4	1703.6	1897.6	2113.8	2354.5
		Due to IRI	-	19.0	19.0	21.0	21.0	21.0	21.0	21.0	21.0	30.0	30.0	30.0	30.0	30.0	30.0	38.0	38.0	38.0	38.0	38.0	38.0
		Due to Rutting	-	37.0	37.0	38.0	38.0	38.0	38.0	36.0	36.0	37.0	37.0	37.0	37.0	37.0	38.0	38.0	38.0	38.0	38.0	38.0	38.0
		Due to Texture	9.6	9.7	9.8	10.0	10.1	10.2	10.4	10.5	10.7	10.8	11.0	11.1	11.3	11.5	11.7	11.9	12.1	12.3	12.5	12.7	12.9
		Due to Waterfilm	484.4	532.4	585.1	643.0	706.6	776.5	853.4	937.8	1030.6	1132.6	1244.7	1367.9	1503.3	1652.0	1815.5	1995.2	2192.6	2409.6	2648.1	2910.2	3198.2
			-	304.0	338.5	377.0	419.9	467.6	520.8	580.0	646.0	719.6	801.5	892.7	994.3	1107.4	1233.5	1373.9	1530.3	1704.5	1898.5	2114.7	2355.4
	Fuel Consumption (l/100km)	**Due to Texture**	689.1	688.6	688.1	687.5	687.0	686.5	686.0	685.5	685.0	684.5	683.9	683.4	682.9	682.4	681.9	681.4	680.9	680.3	679.8	679.3	678.8
		Due to IRI	643.9	644.2	644.5	644.8	645.2	645.5	645.8	646.1	646.5	646.8	647.1	647.4	647.8	648.1	648.4	648.7	649.0	649.4	649.7	650.0	650.3
	Traffic Noise	**Due to Texture** (SMA & DAC)	74.9	74.8	74.8	74.8	74.7	74.7	74.7	74.6	74.6	74.6	74.5	74.5	74.5	74.5	74.4	74.4	74.4	74.3	74.3	74.3	74.2
		Due to Texture (PAC)	74.9	75.2	75.5	75.8	76.1	76.4	76.6	76.9	77.2	77.5	77.8	78.1	78.4	78.7	79.0	79.3	79.5	79.8	80.1	80.4	80.7

Figure D-1: Numerical results of the final model (third worksheet).