STELLINGEN

behorende bij het proefschrift

Permanent deformation in concrete block pavements

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1 december 1997
1
Omdat stenen onder een last t.o.v. elkaar bewegen mag de betonsteenlaag niet als een homogene continue laag met een "equivalente stijfheid" worden gemodelleerd.

2
Loskorrelige materialen kunnen geen (of slechts zeer geringe) trekspanningen opnemen. Het principe van de "buigende laag", waarop veel verhardingen worden ontworpen, is hierdoor per definitie een foute benadering voor het analyseren en ontwerpen van verhardingen waarin ongebonden lagen een belangrijke dragende functie hebben.

3
Langsonvlakheid in combinatie met de dynamische voertuigkarakteristieken zijn van wezenlijke invloed op het gedrag van betonsteenverhardingen en moeten dus in een ontwerpmethodiek worden meegenomen.

4
Het gebruikelijke last equivalentie concept is fundamenteel onjuist voor verhardingen waarin ongebonden materialen een belangrijke functie hebben. In analogie met het spanningsafhankelijk gedrag van ongebonden materialen zou men een lastafhankelijk gedrag van dergelijke constructies moeten definiëren.

5
Binnen de internationale wegenbouw bestaat te weinig "drive" om de realiteit beter te begrijpen, te beschrijven en te beheersen. De drang om complexe zaken te eenvoudig weer te geven overheerst.

6
De enige weg om de kennis rond het gedrag van wegen te vergroten is de weg van de theorie. Zelfs al wordt deze ontwikkeling geroemd door de ganebare werkwijze, zii is door de ontwikkeling van beter inzicht in materiaalgedrag en de komst van steeds meer rekenkracht niet te stoppen.

7
De komst van een verenigd Europa is een stimulans om het systeem van "Rationeel verhardingsbeheer" uit te breiden met een systeem van "Rationeel wegtransport" tot een systeem van "Rationeel wegbeheer".

8
Engineering judgement is een kunde die voorbehouden is aan mensen die over de nodige theoretische kennis en ervaring beschikken.
Moderne auto-ontwerpen maken duidelijk dat voortgang in techniek niet altijd leidt tot een hogere graad van schoonheid, maar veelal tot een eenheidsworst.

In een parlementaire democratie zoals de Nederlandse wordt de publieke opinie en daarmee de regeringssamenstelling in hoge mate bepaald in de redactie kamers van de nieuws-media.

Door aan te dringen op mentaliteitsverandering gaat de overheid haar verantwoordelijkheden uit de weg.

Ieder mens heeft het recht zijn leven in te richten op een manier zoals hij dat wenst. Het is de plicht van andere mensen daar respect voor te hebben, dit ongeacht de (religieuze) overtuiging van die ander.

Nederlanders zien hun land als een grote modelspoorbaan, hetgeen inhoudt dat Nederland nooit af zal zijn.

---

Since under a load blocks move in relation to each other, the concrete block layer may not be modeled by a homogeneous, continuous layer with an "equivalent stiffness"

Granular materials can resist no (or only very limited) tensile stress. The principle of "bending layers", which forms the basis for pavement design, is thus per definition a wrong approach for the analysis and design of pavements in which granular layers have a significant contribution to the bearing capacity.

Longitudinal unevenness in combination with the dynamic characteristics of vehicles are of major importance for the behaviour of concrete block pavements and should thus be considered in a design method.

The use of load equivalency concepts is fundamentally wrong for pavements in which
granular materials have a major role. In analogy with the stress-dependent behaviour of granular materials a load-dependent behaviour of such pavements should be defined.

5
Within the international paving world there is a lack of "drive" to understand, describe and control reality. The urge to over simplify complex matters is too dominant.

6
The only way to increase the knowledge about pavement behaviour is theory. Even when this development is slowed down by common practice, it cannot be stopped due to the development of better insight into material behaviour and the availability of fast increasing computational power.

7
The unification of Europe is a stimulant to extend "Pavement Management Systems" with "Road Transport Management Systems" to "Road Management Systems".

8
Engineering judgement is a skill which is reserved to people that have the required theoretical knowledge and experience.

9
Modern car-designs show that technical progress don't always lead to a higher degree of beauty, but often to a uniformity in designs.

10
In a parliamentary democracy such as the Dutch the public opinion and as a result of that the composition of the government are for a major part determined in the redactional rooms of the news media.

11
By insisting on a change in mentality the government is not taking her responsibilities.

12
Each human being has the right to live the life he chooses. It is the obligation of other people to respect this, regardless of the other's (religious) conviction.

13
The Dutch see their county as a huge model railway track, which implies that the Netherlands will never be completed.
Permanent deformation in concrete block pavements
Permanente vervorming in betonsteenverhardingen

PROEFSCHRIFT

ter verkrijging van de graad van doctor
aan de Technische Universiteit Delft,
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in het openbaar te verdedigen ten overstaan van een commissie,
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Printed in The Netherlands
To my parents Geertruida Maria Huurman-Huijs and Pieter Johannes Huurman:

I will never forget you and will always love you.
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Summary

M. Huurman, Permanent deformation in concrete block pavements (1997)

In this dissertation the results are presented of an extensive research program on the development of permanent surface deformations in concrete block pavements. Due to traffic influences permanent deformations in both the transversal direction (rutting) and longitudinal direction (roughness or longitudinal unevenness) will develop which, when they have developed to a significant extent, cause hinder to the public because traffic safety, driving comfort and aesthetics have deteriorated to unacceptable levels.

The analysis of concrete block pavements is complex since the top layer, made of blocks, is certainly not a continuous, homogeneous layer. The behaviour of this layer strongly depends on the behaviour of the sand filled joints between the blocks. Furthermore the behaviour of the bedding sand layer placed immediately under the block layer is stress dependent. The same holds for the unbound base, made of e.g. a mix of crushed masonry and crushed concrete, which is placed between the bedding layer and the sand sub-base in case heavy loads are expected. All this means that application of the well known multi layer theory for homogeneous materials is not appropriate for this pavement type.

Therefore this dissertation describes, after some introductory chapters (1 to 3), in chapter 4 the research that has been done on the development of a finite element model for the resilient analysis of this type of pavement structure which allows a number of its specific features to be taken into account.

It is clear that no model is capable of generating realistic results if the input is incorrect. The next step therefore was the analysis of a number of in-service block pavements in order to determine the stiffness of the joints. The data required for this analysis have been obtained by means of deflection measurements. The analysis itself is described in chapter 5.

Modelling of the resilient and permanent deformation behaviour of sub-base sands and base materials have also received considerable attention. This was necessary since the stress dependent models that are commonly used in pavement engineering to describe the behaviour of these materials were not good enough to be used for the design of concrete block pavements. Especially the research that has been done on a number of sands is of importance. This is described in chapter 6.

In the chapters 7 and 8 the information of the previous chapters is
brought together in the development of a calculation procedure for the permanent strains that develop in the unbound materials and for the permanent transversal deformation (rutting) that is visible at the pavement surface. Based on extensive calculations using the results of the material research as input, a simplified design model was developed which allows the development of rutting to be calculated as a result of a spectrum of axle loads, tire pressures, materials used, layer thicknesses and the amount of lateral wander of traffic.

Because pavements are never completely smooth, dynamic loads develop which result in a varying rut depth with length. This varying rut depth effects the longitudinal profile in the wheel tracks and thus results in a further increase of the dynamic loads and thus in a further increase of the variation in rut depth and a further decrease of driving comfort. The development of the dynamic loads and its consequences on the development of transversal and longitudinal deformations is extensively described in chapter 9 and illustrated by means of examples in chapter 10.

Verification and validation is the subject of chapter 11. Transversal and longitudinal roughness development as predicted by the models was compared with the deformation behaviour of a number in situ pavements. It was concluded that the models simulate real life pavement conditions quite well. Some differences that occurred could be explained rather well.

In the last chapter of this dissertation, chapter 12, some conclusions are drawn. The most important practical conclusions are as follows:
1. The dynamic interaction between pavement and vehicles results in a decrease of the design life. Vehicle dynamics should be taken into consideration in the design of block pavements for important roads.
2. Regular controls on the technical state of heavy vehicles is of importance in order to overcome excessive pavement deterioration due to poorly maintained vehicles.
3. Block pavements should be laid as smooth as possible to minimize dynamic vehicle responses and so maximize the design life. For similar reasons the bearing capacity of a pavement should vary as little as possible over the length.
4. Decreasing the amount of lateral wander negatively effects pavement performance. This should be considered when designing traffic measures.
5. The deterioration of block pavements with a high bearing capacity is mainly affected by the heavy wheel loads. However on light structures the effects of light wheel loads can not be ignored.
6. Heavy traffic can deteriorate block pavements in residential streets in a relative short period of time. This aspect should be taken into account when
for instance rerouting bus routes.
7. It was shown that different types of sand that all passed the specifications
   can result in a completely different pavement behaviour. Appropriate material
testing is thus needed in order to come to reliable performance predictions.
8. The linear-elastic multi-layer theory is not appropriate for analyzing
   concrete block pavements.
Samenvatting
M. Huurman, Permanente vervorming in betonsteenverhardingen (1997)

In deze dissertatie worden de resultaten gepresenteerd van een uitgebreid onderzoek naar permanente vervormingen in betonsteenverhardingen. Door verkeersinvloeden kunnen permanente vervormingen in zowel de dwars- (spoorvorming) als de langsrichting (langsonvlaktheid) ontstaan. Wanneer deze vervormingen een zekere omvang bereiken, veroorzaken zij hinder doordat de verkeersveiligheid, het rijcomfort en de esthetiek van de verharding afnemen tot een onacceptabel niveau.

De analyse van betonsteenverhardingen is zeer complex. Ten eerste omdat de toplaag van betonstraatstenen zeker niet continu en homogeen is. Het gedrag van deze laag wordt sterk beïnvloed door het gedrag van de met zand gevulde voegen tussen de stenen. De straatlaag direct onder de stenen vertoont bovendien spanningsafhankelijk gedrag. Hetzelfde geldt voor de ongebonden fundering, veelal een mengsel van gebroken metselwerk en gebroken beton, die tussen de straatlaag en het zandbed wordt aangebracht indien grote lasten worden verwacht. Dit alles betekent dat de veel gebruikte meerlagen theorie voor homogene materialen niet toegestaan kan worden voor de dimensionering van betonsteenverhardingen.

Daarom wordt in hoofdstuk 4, na enige inleidende hoofdstukken (1 t/m 3), het onderzoek beschreven dat is gedaan naar de ontwikkeling van een eindig elementen model voor spanningsafhankelijke elastische analyses van dit type verharding.

Het is duidelijk dat geen enkel model reële resultaten oplevert wanneer de input niet reëel is. De volgende stap in het onderzoek was daarom de analyse van een aantal betonsteenverhardingen om de stijfheid van de voegen te bepalen. De benodigde data zijn verkregen door middel van deflectiemetingen. De analyse zelf is in hoofdstuk 5 beschreven.

De modellering van het elastisch en het permanent vervormingsgedrag van zand en funderingsmateriaal heeft de nodige aandacht gekregen. Dit was noodzakelijk omdat de spanningsafhankelijke materiaalmodellen die in de wegenbouw gebruikelijk zijn, niet toereikend zijn bij het ontwerp van betonsteenverhardingen. In het bijzonder het onderzoek dat is verricht aan een aantal typen zand is van groot belang. Dit wordt beschreven in hoofdstuk 6.

In de hoofdstukken 7 en 8 is de informatie uit de voorgaande hoofdstukken gecomcombineerd en is een rekenprocedure ontwikkeld waarmee het ontstaan van permanente rekken in loskorrelige materialen en het ontstaan van
spoorvorming kan worden bepaald. Op basis van een groot aantal berekeningen, met de resultaten van het materiaalonderzoek als input, is een versimpeld model ontwikkeld. Dit model geeft de ontwikkeling van dwarsvlakheid als functie van een spectrum van aslasten, bandenspanningen, gebruikte materialen, laagdikten en de mate van versporing van het verkeer.

Omdat geen enkele verharding volledig vlak is, ontstaan dynamische aslasten waardoor een spoordiepte ontstaat die over de lengte varieert. Deze variërende spoordiepte leidt tot onvlakheid van het langsprofiel in de wielsporen en resulteert dus in een toename van de dynamische aslasten en een afname van het rijcomfort. De ontwikkeling van dynamische aslasten en de consequenties hiervan op de ontwikkeling van dwars- en langsonvlakheid worden beschreven in hoofdstuk 9 en door middel van voorbeelden toegelicht in hoofdstuk 10.

Verificatie en validatie is het onderwerp van hoofdstuk 11. De dwars- en langsonvlakheid zoals die voorspeld worden met behulp van de modellen is vergeleken met de onvlakheid, waargenomen op een aantal verhardingen. Geconcludeerd is dat met de modellen de werkelijke condities goed kunnen worden gesimuleerd. De enkele verschillen die optraden konden gemakkelijk worden verklaard.

In het laatste hoofdstuk van deze dissertatie, hoofdstuk 12, worden enige conclusies getrokken. De voor de praktijk meest belangrijke zijn de volgende:
1. De dynamische interactie tussen verharding en voertuig resulteert in een afname van de levensduur. Bij het ontwerp van betonsteenverhardingen voor belangrijke wegen moet hiermee rekening gehouden worden.
2. Het regelmatig controleren van de technische staat van zware voertuigen is van belang om extreme schade-ontwikkeling van de verharding ten gevolge van slecht onderhouden voertuigen te voorkomen.
3. Betonsteenverhardingen moeten zo vlak mogelijk worden aangelegd om dynamische voertuigreacties te minimaliseren en de levensduur te maximaliseren. Om vergelijkbare redenen dient ook de draagkracht van de verharding zo weinig mogelijk over de lengte te variëren.
5. De schade-ontwikkeling in betonsteenverhardingen met grote draagkracht wordt hoofdzakelijk bepaald door de zware wiellasten. Bij lichte betonsteenverhardingen spelen echter ook de kleinere wiellasten een rol.

7. Aangetoond is dat verschillende typen zand, die allen aan de vigerende specificaties voldoen, resulteren in totaal verschillend constructiedrag. Uitgebreid materiaalonderzoek is dus noodzakelijk om het verhardingsgedrag betrouwbaar te kunnen voorspellen.

8. De lineair elastische meerlagen theorie is niet geschikt voor de analyse van betonsteenverhardingen.
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Introduction and scope of this study

1.1 INTRODUCTION

Over the years a lot of experience in small-element paving has been developed in the Netherlands. With the introduction of the rubber air tire this experience mainly concentrated around clay brick paving. After the second world war the concrete paving block was introduced in the Netherlands. These rectangular blocks had dimensions that closely resembled the dimensions of the clay bricks applied before the second world war, so that the experience in clay brick paving could easily be adopted for concrete block paving.

As a result of the enormous experience in small-element paving, theoretical design methods for concrete block pavements were hardly developed in the Netherlands.

The first international conference on concrete block paving, Newcastle-Upon-Tyne, U.K., 1980, was an eye-opener for the Netherlands. In most other countries there was hardly any experience with concrete block paving. In some of these countries however theoretical design procedures were under development in order to support the application of concrete block paving. Engineers involved in concrete block paving in these foreign countries, at that moment, were not yet fully aware of the complete capabilities of concrete block pavements, but in contrast with their Dutch colleagues they were able to explain parts of the concrete block pavement behaviour.

After the first international conference on concrete block paving the concrete block society in the Netherlands came to the conclusion that a very strange situation had developed: The Netherlands had a strong name in applying concrete block pavements, but compared to other countries there was only limited theoretical understanding on the behaviour of this type of pavement. If the Netherlands wanted to keep on playing an important role in concrete block paving it had to develop more advanced techniques for concrete block pavement design.

The Delft University of Technology initiated a research program into
concrete block paving. This research program was executed in close cooperation with the "working group D3" of C.R.O.W which had to develop a design method for concrete block pavements for Dutch conditions. Graduate students were attracted to develop structural models for the analysis of concrete block pavements (the author was one of these students).

Due to the joint research of the Delft University of Technology and the "working group D3" in the 1980's the knowledge of concrete block pavement behaviour in the Netherlands was strongly boosted upward. Until now students graduate on concrete block pavement behaviour on a regular basis.

After the "D3" research was completed the current research program was initiated to upgrade the design system that was available then. The reasons for upgrading where as follows:

a. In the D3-method a rather straight forward relation was assumed between permanent deformation behaviour and resilient behaviour of granular materials. It was agreed that the upgraded method should take into account real material behaviour since it was realized that within e.g. the category of bases of crushed masonry and crushed concrete there was a wide variety of mechanical behaviour. It was realized that these different characteristics should be taken into account in order to explain the differences in behaviour observed in practice.

b. The D3-method does not take into account longitudinal unevenness. It was agreed that this important type of defect, as long as it is caused by traffic, should be taken into account in an upgraded design model.

1.2 CONCRETE BLOCK PAVEMENTS AND THEIR STRUCTURAL UNIQUENESS

Road engineers mostly consider pavements as being a set of layers placed on top of each other. The toplayer or layers mostly are continuous layers made from a bound material (asphalt or concrete). The design of these pavements basically comes down to checking if the bound toplayers offer enough protection to the unbound layers deeper in the pavement structure and the subgrade. If this is the case then the pavement design is based on the determination of the fatigue life of the toplayers. If the toplayers fail, i.e. show cracking, the protection they offer to the underlying layers decreases. This might lead to failure of the complete pavement structure and thus determines the design life.

In this description things are presented very simplified. The basic problem in concrete block pavement design is however strongly clarified by this simplified presentation.

By far most Dutch concrete block pavements do not have any bound
layers. These pavements can thus not be designed by determination of the fatigue cracking life of any bound layer. A concrete block pavement is in fact already completely "cracked", even directly after construction.

Another problem is that the tools used in the design of pavements in which continuous bound layers are applied can not be used for the analysis of concrete block pavements. These tools (i.e. multi layer programs) can not handle layers in which discontinuities are present as is the case in the concrete block layer.

The design of concrete block pavements thus requires a total different approach than the design of most other pavement types.

The unique structural properties of (Dutch) concrete block pavements (i.e. the lack of bound layers and the discontinuous toplayer) thus make it necessary to develop a complete new approach for their design.

In this research first of all a model for the structural analysis of this type of pavements was developed. This model respects the discontinuities of the toplayer and the stress-dependent behaviour of the underlying granular layers. The model gives insight into the stresses that develop in the unbound layers due to a wheel loading. On the basis of these stresses the development of permanent strain in the various unbound materials in the substructure can be determined. Again this is done by respecting the behaviour of the various materials.

Knowing the development of permanent strain the development of permanent surface deformation can be obtained. The design life of a concrete block pavement is now determined by the maximal allowable rut depth in the pavement.

Another unique property of concrete block pavements is their roughness. In the Netherlands concrete block pavements are mostly constructed by hand and sometimes in a mechanical way. Directly after construction such a pavement will show a certain degree of longitudinal unevenness. Due to this longitudinal unevenness the axle loads that are applied to the pavement vary over the length. As a result the wheel load induced rut depth will also vary over the length of the pavement. A complex dynamic interaction between pavement and traffic is the result.

This interaction is of importance for concrete block pavement behaviour. To get insight into this interaction one can use repeated vehicle simulations. Such an approach is however very time consuming and by far not practical. Therefore also a more convenient model describing the dynamic interaction between traffic and pavement was developed which allows unevenness predictions to be made in a relatively simple way.
1.3 LIMITATIONS OF THIS STUDY

Concrete block pavements can be damaged by numerous causes. The block layer can be damaged by mechanical wear due to for instance the repeated impact of the steel forks of a fork-lift truck. Also chemicals might damage the blocks. In an area of heavy settlements the surface of a block pavement can easily develop deformation due to uneven settlements.

Concrete block pavement damage of the types as described above are not considered in this research, only the damage introduced by normal road traffic is taken into account. Normal public road traffic can however introduce numerous kinds of damage to a concrete block pavement. An example of such damage is that the edge restraints might collapse, resulting in a loss of block pattern. Another form of traffic induced damage is spalling of the concrete block edges. Also shear failure type of rutting can locally develop due to saturation of the substructure.

The previously mentioned types of traffic induced damage are all a result of an improperly designed or constructed block pavement. Failure of the edge restraint can be prevented by applying an edge restraint that is heavy and strong enough. The loss of block pattern and spalling of the edges is prevented by ensuring that the pavement has narrow sand filled joints. Finally good drainage can easily prevent saturation of the substructure.

In this research the behaviour of properly constructed concrete block pavements is considered. In such pavements the most important type of traffic induced damage is permanent surface deformation (i.e. rutting).

Another type of normal road traffic induced damage is longitudinal unevenness. The development of ruts in concrete block pavements is not constant over the length of the pavement. This is caused by two things. First of all the rutting behaviour of the pavement might not be constant over the pavement length. The same wheel load will at some points in the pavement introduce a larger rut depth than at other points.

This varying rutting behaviour can find its cause in varying layer thicknesses in the substructure. Also the quality or compaction of the materials in the substructure might vary over the length of the pavement, as might the subgrade quality. Of course also the quality of the toplayer (i.e. joint stiffness) might vary. It is clear that all these variations affect the rutting behaviour and thus might result in a rut depth varying over the chainage.

This kind of rut depth variations are not considered in this research program. The pavements considered in this research are all assumed to be completely homogeneous in their rutting behaviour over the length of the pavement.

They might however show initial longitudinal unevenness, which brings
us to the second cause of a rut depth varying over the length of the pavement. As a result of this initial longitudinal unevenness dynamic wheel loadings will develop, introducing a non constant wheel load to the pavement. The rut depth variations that are a direct result of this traffic-pavement interaction are considered in this research. The effects of this interaction on both rut development and the development of longitudinal unevenness will be considered.

1.4 ORGANISATION OF THIS DISSERTATION

This dissertation has to serve two goals. First of all it has to give insight into the theories and models developed to get a better understanding of concrete block pavement behaviour. Secondly the knowledge that follows from this insight has to be made accessible for road engineers in practise. To serve both goals this dissertation contains theoretical parts as well as practical parts.

Of course the dissertation starts with some general information about concrete block pavements. In the second chapter of this dissertation the history of small-element paving is described. In this chapter also the state of the art is discussed. Furthermore the chapter discusses the advantages and disadvantages of concrete block pavements over other types of pavement.

The third chapter of this dissertation deals with the basic concept of concrete block pavement performance. In this chapter the general outline of the research program is discussed without becoming very theoretical or detailed.

After the first three introducing chapters, three theoretical chapters follow. The first hereof, chapter four, deals with the development of a structural model for the analysis of the resilient concrete block pavement behaviour. This model is capable in handling the stress-dependent resilient properties of the materials used in the substructure of a concrete block pavement, as well as the specific geometric and mechanical characteristics of the block layer itself.

The properties of the joints interconnecting the concrete blocks in the toplayer of course effect the behaviour of the concrete block layer. In order to obtain the proper concrete block layer behaviour seven Dutch in-service concrete block pavements are considered. Chapter five describes how the joint stiffnesses are determined from the resilient behaviour of these seven pavements.
In order to obtain insight into the properties of the materials used in the substructure of concrete block pavements, laboratory tests are performed on eight sands and four base course materials taken from in-service concrete block pavements. The results of these laboratory tests are discussed in chapter six.

All the properties of the man-made pavement structure and the materials used in it are now known. This enables the resilient analysis of any block pavement loaded by any wheel load.

In chapter seven, several of these resilient analyses are discussed. In this chapter it is explained that the results of such a resilient analysis can be used to determine the development of permanent strains throughout the substructure of a concrete block pavement, provided that the permanent strain behaviour of the used materials is known. It is explained that these permanent strains of course lead to permanent surface deformation. By computing this permanent surface deformation the rutting behaviour of a certain concrete block pavement trafficked by a certain single type of wheel load is obtained.

Real traffic of course consists of numerous different types of wheel loads. To cope with this situation the more practical chapter eight discusses the development of a rutting performance model that is capable in handling the situation in which a pavement is trafficked by various types of vehicles, i.e. wheel load types. Numerous rutting calculations of concrete block pavements are made in which various wheel load types and various substructure designs are considered. By means of regression analyses on the obtained results a more general rutting performance model was obtained.

This rutting performance model gives the rutting behaviour of a concrete block pavement as a function of the substructure design, the quality of the used materials and the properties of traffic. The rutting performance model is capable in giving the complete rutting behaviour of a pavement under the traffic that is using the pavement. This model thus enables the design of concrete block pavements on the basis of any rutting failure criterion.

On the basis of the rutting behaviour, repeated vehicle simulation can give insight into the development of a rut depth varying over the chainage as a result of initial longitudinal pavement unevenness. This is discussed in chapter nine. This repeated vehicle simulation is very time consuming and as a result far from being practical. A relatively simple model was therefore developed that allows to analyze the effect of dynamic traffic loadings in a very short period of time.

The combination of the rutting performance model and the roughness model now enables the computation of the pavement's rutting behaviour under
traffic that shows dynamic reactions to longitudinal pavement unevenness. As a result, insight into the development of both rutting and longitudinal unevenness is obtained. The design of concrete block pavements on the basis of the two kinds of unevenness is now possible.

Some examples of such designs are given in chapter 10. In this chapter it is shown that concrete block pavements will indeed develop transversal and longitudinal unevenness as a result of traffic induced damage.

In chapter 11 the longitudinal profiles of seven in-service concrete block pavements are analyzed. Furthermore it describes how the developed models were verified and validated. This was done by comparing the trends in the rutting and roughness behaviour as predicted by the models with the behaviour observed in the field. These analyses showed that the effects of traffic on the development of roughness as explained by the models are also found in real pavements. Therefore it is believed the models allow reliable performance predictions to be made.

After chapter 11 this dissertation comes to an end with chapter 12 in which the conclusions from the research discussed in the earlier chapters are presented and some recommendations are given.
Concrete Block Pavement Performance, the state of the art

2.1 INTRODUCTION

A public network of maintained paved roads is of enormous economical and social importance for a modern society. In the Netherlands about 80 percent of all commercial cargo, expressed in tons kilometres, is transported by road. For the transport of people, expressed in man-kilometres, an even higher percentage of over 90 percent of all transport takes place by road (1).

In 1996 the public road network in the Netherlands had a length of 124064 km of which 113351 km was paved (2). The replacement value of this network equalled about 100 milliard” Dutch guilders in 1995 (1).

The public road network needed to ensure the transport of goods and people represents enormous annual investments in terms of money and material. Given the importance of a good road infrastructure, the substantial costs for road construction and reconstruction are socially accepted. In 1995 about 4.16 milliard’ guilders has been spent on the public road network in the Netherlands, of which about 1.6 milliard’ guilders were spent on maintenance works whereas about 2.56 milliard’ guilders were spent on total reconstruction of existing roads and construction of new roads (3).

In this chapter the important place of concrete block pavements in the Dutch public road network will be discussed. First of all it will be shown that in other parts of the world small-element paving is a very old means of paving roads and paths, even applied before the area that is now known as the Netherlands became populated. Hereafter the history of small-element paving in the Netherlands is discussed.

* To prevent any misunderstanding it is stated here that 1 milliard guilders stands for 1000,000,000 guilders. At the end of 1996 1 Dutch guilder equalled about 0.58 US dollar or about 0.46 ECU.
It will be shown that a substantial part of the public network of paved roads in the Netherlands consists of concrete block pavements. Attention will be paid to the size of the area paved with concrete blocks and the type of roads on which they are used. Also the current Dutch design method, developed by the C.R.O.W-working group "D3" will be discussed. Attention is also paid to the most occurring defects of Dutch concrete block pavements.

2.2 FIRST APPLICATIONS OF SMALL-ELEMENT PAVEMENTS

Since the beginning of mankind there is a need for paved roads and paths for easy transportation of men and goods. Therefore road engineering is one of the oldest professions known to men. The technique of building roads furthermore was an important technique as is shown for instance in Egypt around 2500 BC (4).

For building the pyramids hundreds of kilometres of roads had to be constructed (4). Until now it is not known for sure how the pyramids were built. It is still not exactly known how the enormous stone building blocks were transported upwards to be placed in their final position. One of the accepted theories says that the Egyptians build ramps over which the building blocks were pulled upwards. Clay bricks, found near the pyramids, seem to confirm this theory and they are believed to be part of the remains of these ramps (5).

Trading largely depends on the ability to transport goods from one place to another. The people in the Far East depended on trading and constructed numerous roads because of this, mostly using stone elements. The remains of some of these roads can still be found. It is for instance known that the city of Babel had paved roads (4).

Besides their commercial value, roads have always been of special interest for the military. A good example of a road especially constructed for military purposes is the "Kings road" constructed by Dareios I (521-486 BC). This road connected the city of Sardes with the city of Soesu (5).

Paved roads also served social purposes in the past, which is perhaps best shown by the Greek. The Greek had many temples and constructed large traffic squares for the temple visitors. These squares were paved using large rectangular flat stones. Of course the construction of these pavements asked for larger investments. Since everyone profited from the paved squares the city of Athens even collected road tax to bring in the money needed to construct the paved squares (4).
After the Greek the Roman empire evolved. During the Roman empire an enormous road network was constructed. This road network had an overall length of about 80,000 km and covered the Roman occupied territories in Europe (amongst others the area that is now known as the Netherlands), Asia and Africa. Of course the centre of this Roman network, mainly constructed for military reasons, was Rome which is founded in 753 BC (4, 6).

One of the first roads constructed by the Romans was the Via Appia which at first connected Rome with the city of Capua. This road which followed the mediterranean coast was for a large part constructed by the military itself and had a width of 4 m to 12 m. The Via Appia remained unpaved until the second century AD. At some points the pavement structure had a total height of more than two meters and consisted of hand placed cemented cobblesstones. The surface of the pavement was formed by flattened stones to ensure comfort. Most paved Roman roads had a surfacing of either rectangular or polygonal flattened stones, see figure 2.1.

Another example of a road constructed by the Romans is found in Temgad in Algeria. Here the Romans used blue/grey rectangular blocks with a length of about 1 m and a width of about 0.25 m to construct the pavement. The remains of this road are still intact (4).

Like the Egyptians the Romans also used clay bricks and tiles for paving. In most cases these burned clay elements were used indoors, however in some cases clay elements were applied on roads (7).

![Image](image.jpg)

*fig 2.1 Example of a Roman paved road in Salonae (Split, Croatia) residential palace of Diocletianus.*

After the collapse of the Roman empire in 476 AD a long period started in which the Roman roads were not maintained and thus deteriorated. Trading in this period mainly took place by means of ships so that there was no commercial reason to invest in roads either. This period lasted until the beginning of the 18th century.
2.3 HISTORY OF SMALL-ELEMENT PAVING IN THE NETHERLANDS

Paths and roads developed because men tends to follow the trails of others. Mankind has always tried to improve the trafficability of these paths and roads by paving them. Sometimes, if the natural surrounding was hardly accessible, paths and roads did not arise from trail. Under these circumstances they had to be constructed (8). For ages, these early pavings consisted of materials which could easily be found in the surrounding area, like cobblestones and small treetrunks (7).

The first Dutch pavements were constructed in the north-east part of the Netherlands, in the provinces of Drenthe and Groningen. In this area trails could hardly develop because of the very poor peat subgrade and the morbid growth of vegetation and therefore peat-roads were constructed. These roads consisted of two treetrunks laid in the longitudinal direction on which treetrunks or planks were laid in the transversal direction so that a road with a width of about 3 m was obtained.

It is not known who built the peat-roads, or why they were built. It is furthermore not known which villages or places were connected by the peat-roads. Some of these peat-roads are however still intact; they were overgrown by the vegetation and disappeared in the peat. In the 19th century they were found as a result of peat cutting. The oldest road through the marshlands is more than 4,000 years old and was located near Nieuw-Dordrecht in the province of Drenthe. This road could only be travelled by pedestrians and could not be used by carts.

The peat-roads improved the bearing capacity of the poor subgrade and made the peat accessible for men. The first carriageway through the peat that could be used by both pedestrians and carts was constructed about 350 BC and was dug up near Valtje in the province of Drenthe in 1936 (8), figure 2.2.

Wooden elements in one form or another were applied for paving roads and paths in the Netherlands for centuries. Small stone elements like cobblestones or stone-setts were not used for paving paths and roads in this country until the 17th century. Only a few stone paved roads were constructed in the Netherlands by the Romans during the first four centuries AD, these roads were only an incident and mainly served the Roman military. After the Romans left in the beginning of the 5th century some of these heavy Roman roads kept being used. The Roman roads were however no longer maintained since there was no government strong enough to organise maintenance.

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Concrete Block Pavement Performance, the state of the art
The area that is now called the Netherlands slowly became more densely populated in the 10th and 11th century. The number of unpaved roads increased and a more or less interlocal road network of unpaved roads developed. This road network remained unchanged for ages. New roads were hardly added to this network until the 19th century. Road improvements were however made in the 17th and 18th century, especially in the old province of Holland (current provinces of north and south Holland) and to a less extent in the provinces of Zeeland and Brabant. At the end of the 18th century, in the Netherlands less than 200 kilometres of road was paved using small stone elements (8).

Compared to surrounding countries the total length of paved roads was very small in the Netherlands. The main cause of the minor Dutch road building activities was the abundant amount of water. Especially in the lower northern and western parts of the Netherlands construction of paved roads was not a priority since travelling by ship was cheaper than travelling by horse and ship travelling furthermore was certainly not slower than travelling by horse and coach. Instead of paving roads the Dutch thus invested in barge-canals.

Investments in roads were only perpetrated in special cases when:
- the construction of a barge-canal was impossible, like the road between Hoorn and Enkhuizen which was finished in 1668,
- local governments could not come to an agreement on constructing a barge-canal, like the road between Rotterdam and Gouda, 1680,
- there were special interests like the demands of the Dutch Royalty, like the road from The Hague-Loosduinen to the castle in Honselersdijk.

At the beginning of the 19th century the Netherlands were placed under French influence, and eventually even became a part of France (1810-1813). As discussed, the Dutch until then had hardly invested in paved roads. As a result the Dutch knew far less about road construction than the French did.
There were no engineers in road construction in the Netherlands, their work was done by surveyors, furthermore skilled paviours were also not at hand. According to reports of the French ministry of foreign affairs (1804) the Dutch were capable in maintaining roads but certainly not in constructing them.

Napoleon wanted his armies to reach the Dutch coast and harbours by road and among other things he planned a paved road from Paris to Amsterdam. He decided that the Dutch had to learn how to build roads by attending the French Ecole des Ponts et Chaussées (8).

The French annexation was too short to complete the network of roads that Napoleon wanted. Also the French knowledge of road building was hardly passed on to the Dutch. The new man in power, the Dutch king Willem I, however learned from the French. He recognised the importance of a good road network and adopted the French road network plans and even expanded them.

At first the roads built by the Dutch were copies of the French roads including the trees planted along both sides of the roads to protect the marching French soldiers against the heat of the sun. Eventually the Dutch however learned by trial and error how to pave roads.

As discussed the Dutch did not develop an interlocal network of paved roads until the 19th century. The history of small-element paving is however mainly concentrated on urban streets. In medieval times these streets served as sewers and also for access. The design and construction of these streets required skill to ensure proper drainage (9).

In the Netherlands first cobblestones have been used for paving streets and roads. This is clearly shown by a painting from the Dutch painter H. de Keyser which shows two paviours constructing a cobblestone urban street, about 1610, see figure 2.3.

To improve the quality of the pavements, stone-setts later became more common. The stone-setts were expensive since they had to be imported because of the lack of stone quarries in the Netherlands. Since clay was found in Dutch river beds, clay bricks were cheaper than the imported stone-setts. The clay bricks could however not resist the steel wheel loads and were thus only applied on foot paths. Apart from the costs, clay brick foot paths furthermore offered more comfort to pedestrians than stone-setts pavements.

Wood-block pavements were used outside the Netherlands since the beginning of the 19th century. These pavements were especially applied where it was desired to reduce the noise from steel wheels and horses’ hooves (9). In the Netherlands however wood-block pavements never became very common. Wood was in the Netherlands mainly applied between and beside the tram rails (8).
fig 2.3  Paviours in the 17th century (a picture of H. de Keyser, about 1610).

More modern forms of small-element paving for roads submerged at the end of the 19th century when the air tyre, invented by Dunlop in 1888, replaced the steel wheels. Clay bricks could now also be used on roads. Special white concrete blocks were often applied in these clay brick roads to create markings for traffic guidance.

At first the substructure of these pavements, if any, was formed by a soil sub-base, consisting of the soil from the ditches which were dug out along the pavement to provide drainage. Later a 100 to 250 mm thick sand sub-base was applied (10).

<table>
<thead>
<tr>
<th>clay brick name</th>
<th>length [mm]</th>
<th>width [mm]</th>
<th>thickness [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Ijssel&quot; format</td>
<td>150</td>
<td>43</td>
<td>65</td>
</tr>
<tr>
<td>&quot;Rijn&quot; format</td>
<td>180</td>
<td>43</td>
<td>85</td>
</tr>
<tr>
<td>&quot;Vecht&quot; format</td>
<td>210</td>
<td>45</td>
<td>105</td>
</tr>
<tr>
<td>&quot;Moppen&quot;</td>
<td>215</td>
<td>43</td>
<td>105</td>
</tr>
</tbody>
</table>

Table 2.1  Most applied early formats of burned clay bricks in the Netherlands (11).

In the early days of clay brick paving, bricks with various dimensions were made and used in the Netherlands, see table 2.1 (11). Through the years it was found that these early formats did not perform as well as the "Waal" format clay brick. The "Waal" format thus became the most applied format, other formats however kept being used. With the introduction of stronger
furnaces it became possible to manufacture thicker "Waal" format bricks called the "dik" format (thick format) and the "klinkerkei", the dimensions of these later clay bricks are shown in table 2.2 (7, 10, 11).

<table>
<thead>
<tr>
<th>clay brick name</th>
<th>length [mm]</th>
<th>width [mm]</th>
<th>thickness [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Waal&quot; format</td>
<td>195</td>
<td>48</td>
<td>85</td>
</tr>
<tr>
<td>&quot;Dik&quot; format</td>
<td>195</td>
<td>64</td>
<td>85</td>
</tr>
<tr>
<td>&quot;Klinkerkei&quot;</td>
<td>195</td>
<td>92</td>
<td>85</td>
</tr>
</tbody>
</table>

table 2.2  The "Waal" format clay brick and the bricks that were derived from it (7, 10, 11).

In 1917 the "N.V. De Vlamovenstraatklinker" was established. This association of clay brick producers (54 plants in 1931) introduced a new clay brick with dimensions that closely resemble the dimensions of the current Dutch concrete blocks. This new clay brick (length x width x height = 200 x 100 x 100 mm³) was called the "Vlamovic" and had spacer nibs on the side surfaces. These nibs guaranteed a certain minimal joint width so that a proper filling of the joints with sand was ensured. In order to prevent spalling, the top surface of the "Vlamovic" had rounded edges.

The "Vlamovic" can not be compared to normal clay bricks. It was designed to have a much larger resistance against wear, spalling and breakage than normal clay bricks so that it could replace the granite stone-sets which were still applied in special cases where clay bricks could not be used. The main difference between normal bricks and the "Vlamovic" is that normal bricks are pressed into shape in wet condition while the "Vlamovic" was pressed from dry material. This special production process made the "Vlamovic" more expensive than normal clay bricks, it was however cheaper than granite stone-sets (11).

In 1930 the "N.V. De Vlamovenstraatklinker" produced 225,000,000 bricks, enough to pave 2,400,000 m² or 400 kilometres of 6 m wide road (12). In 1938 the "N.V. De Vlamovenstraatklinker" was dissolved (11).

The clay bricks were mostly laid in simple patterns like the running bond, the stretcher bond and the diagonal bond. Especially the stretcher bond was very common. Later more complex patterns like the herringbone bond became more popular, see figure 2.4.

With the increase of the weight and number of vehicles at the beginning of the 20th century more attention was given to the substructure of the clay brick roads. The thickness of the sand sub-base increased to 400 or 500 mm. Attention was given to drainage of the sand sub-base in order to prevent saturation of the sub-base or subgrade, and the importance of good compaction was recognized.
If the bottom of the sand sub-base was not above the groundwater level, 400 to 500 mm thick sand sub-bases were not thick enough if the subgrade consisted of clay. In these cases such a thin sand sub-base was pushed into the saturated clay. In order to prevent this, 800 to 1000 mm thick sand sub-bases were applied in case of clay subgrades and high groundwater levels (10).

With the further increase of the weight and number of vehicles a further strengthening of the sub-structure was sometimes needed. This strengthening was at first achieved using base layers made from old or badly shaped clay bricks. Sometimes this single brick layer base was simply formed by the old
pavement which was in these cases covered with about 50 mm bedding sand and then repaved using new clay bricks. When this single clay brick base was not sufficient a double brick layer was used as a base, see figure 2.5.

A third type of base was formed by a single brick base layer with a layer of rubble on top of it. The total thickness of this base was about 150 mm (5, 6, 7). The rubble dumped upon the single clay brick layer was very coarse, grain size between 40 mm and 70 mm. This layer was compacted by 8 to 10 ton rollers so that the rubble was partly crushed and a well compacted layer was obtained. It was recognized in those days that the rubble used for the base layer should consist of material with a high crushing resistance, preferably broken clay bricks. The rubble applied for base layers in those days was however often not sorted out. As a result the rubble used in the base layers sometimes consisted of a mixture of hard and soft material. In these cases a poor base was obtained in which weak areas were present. A clay brick pavement with such a poor base layer performed less than a clay brick pavement without any base layer at all.

Bound base layers were far less often applied than the described unbound base layers. The most popular bound base was a stabilized clay base. Such base layers consisted of a sand/clay/water mixture to which 5% to 8% cement was added. This mixture is highly compactable and the quality of the stabilized clay base largely depended on good compaction.

On top of the described base layers a 40 to 50 mm thick sand bedding layer was needed for compensation of the differences in brick dimensions (thickness) and to ensure good bedding.

After the second world war the Dutch population increased substantially. Combined with war damage reconstruction works, this resulted in an enormous demand for clay bricks for new houses. A shortage of building bricks developed and the manufacturers of clay bricks for roads switched their production to clay building bricks.

The concrete block was, under these circumstances, easily accepted as a substitute for the clay bricks. Initially the concrete block and clay bricks had the same dimensions and similar costs, but eventually concrete blocks were produced at just 40 percent of the costs of clay bricks. Besides the costs, concrete blocks also have the advantage of a far better uniformity of shape and dimensions over clay bricks. As a result of these advantages clay bricks became less and less applied. Nowadays clay bricks are mainly used for maintenance of roads and squares in historical city centres.

As a result of the extensive bombing a lot of building rubble was available after the second world war. This rubble proved to be a good base material. The rubble was used uncrushed and heavy compaction machinery
was needed to ensure proper compaction through in-place crushing. Later the rubble was crushed before it was used as a base material. Current Dutch bases are very often made of crushed concrete or a crushed masonry/concrete mix. Crushed masonry bases are hardly used in the Netherlands any more.

The dimensions of the concrete blocks changed. Nowadays the most applied blocks have a length of 211 mm and a width of 105 mm. The thickness of these blocks can vary from 60 to 120 mm, the most common blocks however have a thickness of 70 or 80 mm. Figure 2.6 shows a typical modern Dutch concrete block pavement structure.

![A typical modern Dutch concrete block pavement structure.](image-url)

Special shaped interlocking blocks have been introduced in the Netherlands. These interlocking blocks are very successful in foreign countries, like Australia (2), which adopted block paving from Europe. In the Netherlands however interlocking blocks were never a success and simple rectangular blocks are still by far most common.

This is most likely a result of the skills which are required for constructing a high quality block layer using rectangular blocks with some dimensional tolerances. The Dutch paviour has these skills. In most foreign countries the required skilled paviours are not available, so that the introduction of interlocking blocks in these countries resulted in an increase of the quality of the block layer.
In the 1960’s the last concrete block pavements were constructed on primary roads in the Netherlands. Nevertheless the use of concrete blocks was increasing until the end of the 1970’s as a result of the expansion of built-up areas, the ongoing replacement of clay bricks and the growth in the use of blocks for industrial paving. Since the end of the 1970’s the use of concrete blocks in the Netherlands is more or less stable.

2.4 THE QUANTITY OF CONCRETE BLOCK PAVING IN THE NETHERLANDS

In 1996 the total length of the road infrastructure in the Netherlands was 124064 km, of which 10714 km was unpaved (2). Neglecting the unpaved roads, 5 categories of roads are distinguished in the Netherlands (in decreasing order of traffic intensity):
1. primary roads: state motorways and other important national roads, managed by state government,
2. secondary roads: important roads for regional traffic, in most cases managed by provincial road authorities, sometimes by city governments,
3. tertiary roads: roads for inter-urban traffic, mostly managed by provincial authorities, but in some cases managed by city governments,
4. urban roads: roads for urban traffic, managed by city governments,
5. low volume and other roads: less important inter-urban, rural and farm-to-market roads, managed by city government and polder boards (1).

The three most important pavement types in the Netherlands are asphalt pavements, concrete block pavements and concrete pavements. About 55 to 65 percent of the total amount of paved roads are asphalt pavements, 30 to 40 percent of the paved roads are concrete block pavements and about 3 percent of the paved roads are concrete pavements.

Asphalt pavements are applied for all road categories throughout the Netherlands. Concrete pavements are mainly applied on firm sandy subgrades since these pavements are not flexible and can thus not handle severe uneven settlements. Concrete pavements are mainly used for primary and secondary roads and especially on agricultural roads and cycle paths.

Concrete block pavements in the Netherlands are only applied on low speed roads, maximum speed 50 to 70 km/h at most. It might be clear that these roads are mainly found within built-up areas and in industrial areas. Very rarely concrete block pavements are also used for rural roads.

As discussed concrete block pavements in the Netherlands are mainly used within built-up areas and in industrial areas. The importance of concrete
block pavements for this kind of roads can be shown by considering the paved area of the city of Rotterdam. With its see-port, Rotterdam is the most important city of the Netherlands in economical terms. The city of Rotterdam consists of a combination of "normal" built-up areas and industrial areas.

Including the port industrial area the Rotterdam public works department is responsible for about 25,400,000 m$^2$ of paved area. The breakdown of this total paved area to pavement types shows that in 1994 29% of this area consisted of concrete block pavements, figure 2.7 (13).

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{fig2_7.png}
\caption{Breakdown of the paved area in Rotterdam to pavement type in 1994 (13).}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{fig2_8.png}
\caption{Breakdown of the paved roads in Rotterdam to pavement type in 1994.}
\end{figure}
As shown by figure 2.7, 33% of the total paved area in Rotterdam is paved with concrete flags. These flags are only used on footways and cycle paths and not on roads. Only considering carriage roads the breakdown given in figure 2.8 shows the importance of concrete block pavements for the Rotterdam public road infrastructure. It shows that 43.3% of the paved roads in Rotterdam, expressed in m², is paved by means of concrete blocks.

### 2.5 THE C.R.O.W D3 DESIGN METHOD

The first Dutch design method for concrete block road pavements was developed by the working group D3 of the Dutch Centre for Research and Contract Standardization in Civil and Traffic Engineering, C.R.O.W. It was first presented in 1988 in Rome, at the Third International Conference on Concrete Block Paving, and is extensively described in C.R.O.W-publications (14, 15).

The C.R.O.W design method is mainly based on the assumption that the development of permanent deformation in cohesive and non-cohesive granular layers controls the performance of the pavement, and is proportionally dependent on the vertical resilient deformation of these layers under a wheel load, equation 2.1.

To use this equation, the resilient behaviour of concrete block pavements, \( U_r \), in equation 2.1, has to be known. For this purpose initially eight test sections were constructed, two in Alphen a/d Rijn (peat subgrade) and six in Rotterdam (sandy clay subgrade). Later four extra test sections were constructed at the Europe Combined Terminal (ECT) on the Maasvlakte (sand subgrade).

\[
U_p - U_r = f(N_a)
\]

With

\[
f(N_a) = a + b \log N_a \quad \text{for cohesive materials}
\]

\[
f(N_a) = a \cdot N_a^b \quad \text{for non-cohesive materials}
\]

where:

- \( a, b \): constants depending on the material of a layer [-]
- \( U_r \): resilient vertical deformation of a layer under the centre of a wheel load [mm]
- \( U_p \): permanent vertical deformation as a result of \( N_a \) axle load repetitions [mm]
- \( N_a \): number of axle load repetitions [-]
Extensive Falling Weight Deflection (FWD) measurements were performed on the first eight test sections. The data from these measurements showed that the maximum deflection (in the load centre) of concrete block pavements first remains constant after construction. At a certain number of equivalent 80 kN axle load repetitions \(N_{al}\), a decrease of the maximum deflection develops. At the Delft University of Technology the "Progressive Stiffening" theory was developed which explained the observed decrease of FWD deflections in time \((14, 15)\), equation 2.2.

\[
U_r - d_{\text{max}} \quad \text{for } N_{ae} \leq N_{al}
\]

\[
U_r = p + q e^{-\log N_{ae}} \quad \text{for } N_{ae} > N_{al}
\]

(2.2)

where:

- \(p, q\): constants depending on the material of the layer [mm]
- \(d_{\text{max}}\): initial resilient vertical deformation of the layer under a 80 kN standard axle load [mm]
- \(N_{ae}\): number of equivalent 80 kN standard axle load repetitions [-]
- \(N_{al}\): \(N_{ae}\) at which progressive stiffening begins [-]

By implementing equation 2.2 in equation 2.1 the permanent vertical deformation of any layer in the centre of a wheel load can be determined. Since \(U_r\) is not constant in \(N_{ae}\), equation 2.2 has to be rewritten; this results in equation 2.3.

\[
\frac{dU_r}{dN_{ae}} = U_r + \frac{df(N_{ae})}{dN_{ae}}
\]

(2.3)

For non-cohesive granular materials, combining the equations 2.1 to 2.3 leads to equation 2.4. In a similar way equation 2.5 is found for cohesive materials.

The equations show that six parameters, i.e. \(d_{\text{max}}, p, q, N_{al}, a\) and \(b\), per layer have to be known in order to determine the expected development of rut depth in a concrete block pavement.

\(d_{\text{max}}, p, q\) and \(N_{al}\) describe the resilient behaviour of a layer. The values of \(d_{\text{max}}, p\) and \(q\) of the various layers in the initial eight test pavements were determined on the basis of the results of the FWD. Hereto the contribution of individual layers to the total maximum deflection of the test pavements had to be known. In order to determine the deflections \(U_r\) in the various layers of a substructure use was made of the equivalence theory of Ivanov \((16, 17)\). The value of \(N_{al}\) follows from the values of \(d_{\text{max}}, p\) and \(q\).
Granular materials:

\[
\text{for } N_{ae} \leq N_{al}:
\]

\[
\frac{dU_p}{dN_{ae}} = \frac{d}{dN_{ae}} f(N_{ae}) = d_{\text{max}} a b N_{ae}^{b-1} 
\]

\[
U_p = \int_{0}^{N_{\text{ae}}} d_{\text{max}} a N_{ae}^{b} dN_{ae} = d_{\text{max}} a N_{ae}^{b}
\]

\[
\text{for } N_{ae} > N_{al}:
\]

\[
U_p = d_{\text{max}} a N_{ae}^{b} + \int_{N_{al}}^{N_{ae}} a b N_{ae}^{b-1} \left( p + q e^{-\log N_{ae}} \right) dN_{ae} 
\]

\[
= d_{\text{max}} a N_{ae}^{b} + \int_{N_{al}}^{N_{ae}} a b p N_{ae}^{b-1} + a b q N_{ae}^{-0.4343} \log^{2.7183} N_{ae}^{b-1} dN_{ae}
\]

\[
= d_{\text{max}} a N_{ae}^{b} + \int_{N_{al}}^{N_{ae}} a b p N_{ae}^{b-1} + a b q N_{ae}^{-1-0.4343} dN_{ae}
\]

\[
= d_{\text{max}} a N_{ae}^{b} + a p \left( N_{ae}^{b} - N_{al}^{b} \right) + \frac{a b q}{b-0.4343} \left( N_{ae}^{-0.4343} - N_{al}^{-0.4343} \right)
\]

Cohesive materials:

\[
\text{for } N_{ae} \leq N_{al}:
\]

\[
U_p = d_{\text{max}} (a + b \log N_{ae})
\]

\[
\text{for } N_{ae} > N_{al}:
\]

\[
U_p = \dot{u_{\text{max}}} (a + b \log N_{ae}) + \dot{\sigma} q \log \frac{N_{ae}}{N_{al}} - \dot{\sigma} q \log \left( N_{ae} - N_{al} \right)
\]

On the eight initial test pavements, rut depth measurements were performed, using a 1.2 m long straight edge. For the various materials in these test pavements, i.e. sand, crushed concrete and crushed concrete/masonry mix, the values of a and b could be determined using the results of these rut depth measurements. Using these a and b values the development of the rut depth under a 1.2 m long straight edge was now known as a function of the type of material and the resilient deformation.
Figure 2.9 shows typical results of a C.R.O.W D3 analysis of two of the Rotterdam test pavements (R1 + R2). The figure shows the development of the deflection $U_r$ of the combined 900 mm sand sub-base with the 50 mm sand bedding layer. As shown by the figure progressive stiffening started at 3500 80 kN axle load repetitions. The figure also shows the development of the function $f(N)$, equation 2.1. The combination of this function with the development of $U_r$ results in the rut depth development. The effects of progressive stiffening become clear by comparing the rut depth development on the basis of a constant $U_r$, which equals $d_{max}$, with the rut depth development on the basis of a decreasing $U_r$ according to the progressive stiffening theory.

![Graph showing typical behaviour of a concrete block pavement.](image)

Fig 2.9 Typical behaviour of a concrete block pavement.

The results of the analysis of the test pavements made it possible to determine the development of rut depth in any concrete block pavement, constructed using a sand sub-base only or a sand sub-base together with a crushed concrete or a crushed concrete/masonry mix unbound base. In order to come to the C.R.O.W D3 design method, calculations were made for 84 different pavements. On the basis of the calculated development of the rut depth (under a 1.2 m straight edge) and the adopted rut depth standard of 15 mm design charts were made. One of these charts is presented in figure 2.10.
2.6 MOST OCCURRING DEFECTS

Concrete block pavements can either fail as a result of block layer failure or as a result of substructure failure. A typical type of block layer failure is joint widening, while permanent deformation (rutting) is a typical form of substructure failure. Hereafter both types of failure will be discussed.
Dutch concrete block pavements can show significant resilient deformation under heavy traffic loadings, deflections up to 2 mm and more due to a 50 kN load have been measured. As a result of these large deflections, block translation and rotation occurs. As shown by figure 2.11, blocks are pushed outwards, resulting in high forces on the edge restraint that act to push the edge restraint outward. If the edge restraint is not strong enough only part of this deformation is resilient, resulting in wider joints after a load passage. These wider joints will get filled with soil, sand and dirt, so that an ongoing process of joint widening can develop. Under these circumstances the edge restraints will be pushed outward further and further.

Since wide joints are less stiff than narrow joints, the equivalent stiffness of the block layer decreases, leading to larger stresses in the substructure. Eventually the substructure may fail, resulting in large permanent surface deformation (rutting).

If the substructure of a block pavement in which ongoing joint widening developed, does not fail as a result of the weakening of the block layer, the pavement will eventually fail as a result of a total disruption of the laying pattern of the blocks.

The previously described block layer failure mostly occurs in non built-up areas, where it is often found difficult to create a sufficient edge restraint. In built-up areas problems as a result of insufficient edge restraints are more rare. The concrete blocks are in these cases not only kept together by the edge restraints but also by the sidewalk, located directly besides the concrete block road, see figure 2.12. The sidewalk construction gives the edge restraint additional support, so that it is more difficult to move the edge restraint.

![Diagram](image-url)

**fig 2.12** Surroundings of the edge restraint in a build-up area.

In case of a sufficient edge restraint the described process of joint widening will not occur. In this case, too narrow joints may lead to direct contact between concrete blocks, which can cause spalling of the edges (18, 19). Of course the chance of direct contact between blocks, and thus of spalling of the edges, increases if the deflections become larger. In Belgium
and the Netherlands a special shaped rectangular concrete block with spalling free side surfaces is used in case large deflections are expected (figure 2.13). The shape of this block reduces the chance of direct contact between blocks, so that spalling of the edges is prevented.

![fig 2.13 Special shaped concrete block with spalling free side surfaces.](image)

If the joints between the blocks are narrow, but wide enough to prevent direct block contact (joint width 2 to 3 mm) and the edge restraint is capable in resisting the forces applied on it, then no block layer failure is to be expected. In these cases failure is always a result of substructure failure. This kind of failure can develop very fast in case of overloading of the pavement. Traffic in these cases results in dilatation in the granular substructure. The substructure thus looses density and as a result of that it looses shear resistance. Because of this pavement damage develops faster and faster, resulting in large permanent surface deformations. These structures are not stable and are simple not capable in carrying the traffic loads.

If the granular substructure is strong enough, the block pavement will behave "normally". For pavements with only a sand sub-base this means that permanent strain in the substructure initially develops relatively fast. As the pavement becomes older the rate of permanent strain development becomes smaller and in some cases might even come to a complete stop, see figure 2.9.

The normal or stable rutting behaviour of concrete block pavements with a base might be similar to this behaviour of pavements without a base course. Depending on the base course material it is however also possible that concrete block pavements with a granular base will at the end of their design life show an increase of accumulation of permanent surface deformation, i.e. rut depth (20).

The earlier described defects are all caused by traffic loadings; on poor subgrades however uneven settlements will also lead to surface deformation. In these cases the concrete block pavement might very well be capable in
resisting the traffic loadings but the subgrade is not capable in carrying the weight of the pavement structure itself and settlements occur. On most types of subgrade this process will eventually come to an end. In Dutch peat, unfortunately, the settlements only come to an end after a very long time. Under these circumstances block pavements can fail as a result of uneven settlements, while the traffic induced damage remains acceptable.

2.7 ADVANTAGES AND DISADVANTAGES OF CONCRETE BLOCK PAVEMENTS

In this paragraph the advantages and disadvantages of concrete block pavements are discussed. Concrete block pavements are hereto often compared with the most common pavement type, i.e. asphalt pavements.

Unlike the production of asphalt, the production of concrete blocks does not rely on the availability of expensive petroleum derivatives. The production of concrete blocks only demands suitable aggregates and cements. Furthermore the production of concrete blocks consumes less energy than the production of asphalt.

Blocks with adequate strength and durability can only be produced under high pressure and controlled vibration using a low slump mix. Hereto most blocks are made in mass-producing plants, achieving excellent dimensional and strength tolerances (9).

Unlike asphalt pavements, concrete block pavements can be constructed in a manual way using only the most simple equipment (21). Mechanical laying however is possible and should be stimulated from an ergonomic point of view. This form of laying of course requires special machinery. The costs of this machinery are, depending on the size of the area to be paved, however paid back by a reduction of the costs of labour. In the Netherlands concrete blocks are laid mechanically more and more, especially at larger block paving sites, see figure 2.14.

Another advantage of concrete block pavements is that their construction can take place as long as the substructure is non frozen and not saturated, whereas the construction of asphaltic pavements requires higher air temperatures, resulting in a shorter construction season.
Concrete block pavements have one substantial operational disadvantage when compared with asphaltic pavements, they only offer limited comfort. The main reason hereof is the marginal longitudinal evenness of a concrete block pavement. Even new, hardly trafficked Dutch concrete block pavements only offer limited comfort; permanent deformation introduced by traffic will only make things worse. Because of their higher roughness, concrete block pavements are only applicable for low speed roads (maximum speed 50 to 70 km/h).

Another disadvantage of the poor longitudinal evenness, combined with rutting, is that puddles will form during rain. Of course puddles have a negative effect on the vehicle behaviour on the road and thus affect safety. Furthermore the puddles will lead to an increase of the water penetration which may affect the bearing capacity of the substructure.

The last contribution to discomfort is a result of the production of noise that is produced when a vehicle travels a concrete block pavement. There are however indications that the noise production of concrete block pavements can be reduced by applying blocks without chamfers. Since this topic is beyond the scope of this study it will not be discussed further.

Except for these disadvantages concrete block pavements offer a number of operational advantages. Unlike asphalt pavements, concrete block pavements with an adequate base can be made to resist large static loads, even on hot summer days.
Since most block pavements do not have any bound layers, they easily give access to underlying infrastructure like cables and pipes. Furthermore the blocks can be replaced afterwards, so no road building material is wasted too. Similar to this, repairs of local damage in block pavements are easily performed without the use of expensive equipment and without the loss of road building material (blocks).

Blocks can be manufactured in various colours and can be laid in numerous bonds and patterns, so that infinite possibilities are obtained. This means that the appearance of a block pavement can be designed up to a high degree. Architects sometimes make use of this possibility and create very appealing, colourful pavements.

The last advantage of block pavements over other pavement types is the relatively easy way in which they can be reconstructed. Reconstruction means: taking away the concrete blocks, making repairs on the substructure and laying back the blocks. The blocks which are taken from the pavement hardly require reprocessing, only the joint sand which might stick on the blocks has to be removed; about 90% to 95% of the blocks can be reused.

In the case of settlements the substructure has to be relevelled by adding sand before the blocks are replaced. This of course enlarges the weight of the pavement structure and thus causes additional settlements. In the case of a substructure with a base layer the adding of the sand furthermore results in a thicker bedding sand layer. This of course changes the substructure in such a way that it might have a negative effect on the concrete block pavement (rutting) behaviour.

2.8 CONCLUSIONS

Based on the information discussed in this chapter it is concluded that concrete blocks in the Netherlands became the most widely used elements for small element paving because of their low costs and high dimensional consistency. Concrete block pavements are very important for the Dutch road infrastructure. Within Dutch built-up areas (cities and industrial areas) concrete block pavements can even be considered as the most important pavement type.

A concrete block pavement with sufficient edge restraints and a well-designed substructure will only slowly come to failure and has good operational properties. Such a pavement has numerous advantages over other pavement types. It however has one major disadvantage which is its limited comfort. As a result concrete block pavements are only applicable on low speed roads (maximum speed 50 to 70 km/h).
Basic concept of Concrete Block Pavement Performance

3.1 INTRODUCTION

In this chapter the need for models to predict the development of longitudinal and transversal unevenness in concrete block pavements is described. A conceptional discussion will be given on the type of models needed and their importance for design and maintenance planning purposes.

An overview will be given of the various subroutines which are needed to develop prediction models for transversal and longitudinal unevenness with main emphasis on the necessary input. After this, the objectives of this study are defined and the program that was developed to reach the objectives is described.

3.2 THE BEHAVIOUR OF CONCRETE BLOCK PAVEMENTS

In this section the behaviour of concrete block pavements is described. A distinction is made between resilient deformation behaviour and permanent deformation behaviour.

3.2.1 Resilient deformation behaviour

Like any type of pavement a concrete block pavement consists of several layers. The toplayer of concrete blocks transfers the traffic loads to the substructure of the pavement. In the Netherlands the substructure of concrete block pavements mostly consists of unbound granular materials.

In special cases, like heavily loaded industrial block pavements or block pavements on airports loaded by airplanes, bound base layers are applied. This research program however deals with the behaviour of Dutch concrete block pavements for "normal" road traffic and thus only considers the behaviour of pavements with an unbound granular substructure.
The toplayer of a Dutch concrete block pavement mostly consists of rectangular concrete blocks, length 211 mm and width 105 mm, in herringbone bond. Given the limited horizontal dimensions of the concrete blocks, the individual blocks in the block layer hardly have a load spreading capacity. The block layer in total however obtains a certain load spreading capacity if the individual blocks are interconnected. For this purpose it is not only important that the joints between the blocks remain narrow (2 to 3 mm), they should be filled with sand too.

If a block layer complies to these conditions, a load applied on one block will result in reactive joint stresses. As a result of these joint stresses the load will partly be transferred to neighbouring blocks so that a certain load spreading capacity is obtained.

In figure 3.1 the principle of a load spreading block layer is given. The loaded block is forced down by the load. This results in shear stresses in the joints between the loaded block and adjacent blocks. As a result of these shear stresses adjacent blocks are rotated. This rotation narrows the upper part of the joints, while the bottom part is widened, see figure 3.1.

![load](image)

**fig 3.1** Load spreading in a concrete block layer due to reactive joint stresses.

The sand in the joints can not take tensile stresses, so that the bottom part of the joint becomes free of stresses. In the upper part of the joint however compressive normal stresses develop. In this upper part of the joint now also shear stresses can develop.

Of course reactive stresses also develop in the underlying bedding layer. Under the loaded block stresses will develop over the total block surface. Under the adjacent blocks, however, stresses will mainly develop at some distance from the loaded block. This is caused by the difference between the deflection of the loaded block and the deflections of the adjacent blocks.
It is clear that the load spreading capacity of a concrete block layer completely depends on the ability of the block layer to build up joint stresses. As stated before this requires narrow joints filled with sand. A good edge restraint is however also required. As is shown in figure 3.2 the concrete block layer, when subjected to a wheel load, shows the tendency to expand in the horizontal direction as a result of block rotation. Hardly any joint stresses can develop if this block layer expansion is not restricted by the edge restraint. As a result the load spreading capacity of the block layer will now only be limited.

If the edge restraint is however very rigid then hardly any block layer expansion can occur. In this case block rotation will lead to compression of the jointing sand and thus to joint stresses, limiting the amount of block rotation. Now the block layer will show its maximum load spreading capacity.

*fig 3.2*  
Block layer expansion as a result of block rotation.

Underneath the block layer one or more layers of unbound granular material are present in the substructure of the block pavement. These granular materials all show stress dependent resilient behaviour (22). This stress dependency is of major influence on the stress distribution in the substructure as explained hereafter.

Stiff material tends to attract stresses. If the substructure of a concrete block pavement consists of a material which shows a strong increase of its stiffness with increasing stresses, a very stiff area will develop directly underneath the load where the wheel load stresses are introduced in the substructure. The stresses in this stiff area will now be high. In this case the load spreading capacity of the pavement will be poor.

If the materials applied in the substructure do not show a strong increase
of stiffness with an increase of stresses, the differences in stiffness throughout the substructure will remain small. The stiff area, attracting stresses, underneath the load will not develop now. As a result the substructure shows a better load spreading capacity in which the stresses remain relatively small.

The previous examples clearly show that the stress dependent resilient behaviour of the granular materials applied in the substructure of any pavement seriously effects the stress distribution that will develop under a wheel load.

### 3.2.2 Permanent deformation behaviour

If a concrete block pavement is loaded it will show some deformation. The major part of this deformation is resilient, a small part however is irreversible. As a result of the irreversible part of the deformation under a load, the block pavement will slowly build up permanent deformation due to the repeated traffic loading.

The blocks themselves are made from concrete. Given the enormous stiffness of concrete (Young’s modulus of 35,000 to 40,000 MPa) relative to the stiffness of granular materials (resilient modulus “Mr” of 100 to 1000 MPa) the deformations in the blocks remain very small. Since concrete is of course a bound material it is furthermore expected that most of the minor deformation of the blocks will be resilient. By combining these statements it is concluded that the permanent deformation in the concrete blocks, if any, can be neglected.

All the permanent surface deformation in a concrete block pavement thus is a result of permanent strain development in the substructure and the subgrade. The permanent strain development in granular materials heavily depends on the stresses to which the material is subjected. If the stresses in the material approach the stresses at failure, permanent strain will develop faster. The permanent strains in the substructure will of course lead to permanent surface deformation, i.e. rutting.

A concrete block pavement structure is most likely not homogeneous over the length of the pavement. The thickness of the various layers and the quality of the granular materials (grading, compaction, etc.) will show variations over the length of the pavement. As a result ruts will develop faster at some spots than at other spots on the same pavement structure. The pavement will thus show rutting with a varying depth over the chainage. This variation of rut depth in the longitudinal direction of course affects the longitudinal unevenness in the wheel tracks.

Another reason for the development of longitudinal unevenness are the
effects of dynamic loads. As discussed in chapter 2, even new concrete block pavements show some longitudinal unevenness (roughness). As a result of the initial longitudinal unevenness in a concrete block pavement, axle loads will vary with the chainage. At those places where the dynamic axle load is larger than the average or static load, a larger rut depth will develop. At places where the dynamic axle load is relatively small a smaller rut depth will be found. Dynamic effects will thus also lead to rut depth variations or longitudinal unevenness.

3.3 MODELLING THE BEHAVIOUR OF CONCRETE BLOCK PAVEMENTS

3.3.1 Introduction

In this research project it was decided to approach the behaviour of concrete block pavements in a fundamental way. The basic concept will be, to calculate first of all the stress distribution in the substructure under a wheel load, taking into account the stress-dependent resilient behaviour of the various granular materials applied in the substructure.

Then the permanent stain development in the substructure is calculated taking into account the stress dependent behaviour of the materials and the calculated wheel load stresses. Knowing the permanent strains throughout the substructure, the permanent surface deformation (rutting) can be determined.

3.3.2 Resilient behaviour

In order to calculate the resilient behaviour of a concrete block pavement, a structural model will be needed. Since a wheel load is transferred to the substructure by the concrete block layer, the structural model should at least have a good representation of the block layer in order to come to accurate stresses in the substructure. The model should thus have a discontinuous top layer in which individual blocks can rotate and translate freely. Like in reality, the blocks should be interconnected by joints in which only stresses can develop when the joint is under compression.

The structural model of the substructure can be quite simple. Modelling the substructure is really a matter of applying the proper stiffness distribution. For this reason the stress-dependent resilient behaviour of a granular material has to be described as accurate as possible. As discussed earlier the distribution of stiffness is of major influence on the stress distribution under a wheel load. If accurate resilient material models are at hand then the stress
distribution can be calculated accurately.

The last step in the resilient deformation analysis will be the calculation of the stress ratio, which is the ratio of applied stress to stress at failure, throughout the substructure of the concrete block pavement. Hereto use is made of the well known theory of Mohr-Coulomb.

A discontinuous toplayer can only be modelled using a finite element model. Such a model can furthermore easily handle stress-dependent resilient behaviour. A further explanation of the resilient calculation will be given on the basis of the flow chart presented in figure 3.3.

![Resilient calculation](image)

*fig 3.3 Flow chart of the resilient calculation on concrete block pavements.*

The necessary input data are:

- **Loading**: The stresses in the substructure of course depend on the magnitude of the load that is applied to the block layer. The contact pressure (or the radius of the loading area) of the load is also of influence on the stresses that will develop.
- **Joint behaviour**: The block layer transfers the load to the substructure. The joint stiffness is of course of influence on the load spreading capacity of the block layer and thus on the stresses in the substructure.

- **Resilient behaviour**: The stress-dependent resilient behaviour of granular materials is of large influence on the stress distribution in the substructure. Of all the materials used in the substructure the stiffness as a function of stress, \( M_r = f(\sigma) \), has to be known. The same holds for the Poisson's ratio, \( \nu = f(\sigma) \).

- **Density**: Not only wheel load stresses occur in the substructure, but also dead weight stresses are present. In order to compute these stresses the density of the materials used in the substructure has to be known.

- **Structure**: Of course the layer thicknesses and the materials used in the various layers have to be known.

- **Subgrade**: The properties of the subgrade are of influence on the stresses in the substructure of the block pavement. For reasons of simplicity the subgrade is assumed to have a stress-independent resilient behaviour. As a result the Young's modulus and the Poisson's ratio of the subgrade are essential input parameters.

- **Strength**: In order to translate the calculated stresses into stress ratios (applied stress over stress at failure) it will be necessary to know the stress-dependent strength of the granular materials used in the pavement structure.

The flow chart given in figure 3.3 shows the following procedure. On the basis of the input given above, the values of the resilient modulus and the Poisson's ratio throughout the substructure are estimated first of all in the process "Estimate \( M_r \)-distribution". Given this \( M_r \)-distribution, wheel load stresses are computed; this is the heart of the resilient calculation or finite element model. The dead weight stresses are added to the wheel load stresses, so that the stress distribution in the substructure is obtained. All this is done in the routine "Compute wheel load and dead weight stresses". On the basis of the calculated stresses the proper \( M_r \)-distribution is computed in the process "Compute \( M_r \)-distribution".

Now the estimated \( M_r \)-distribution is compared with the calculated \( M_r \)-distribution. If the estimated \( M_r \)-distribution is accurate enough then the calculated stresses are translated to stress ratios, so that the distribution of the ratio of applied stress over stress at failure throughout the substructure is known. This is done in the process "Compute distribution stress ratio".

It is however most likely that the computed \( M_r \)-distribution does not equal the first estimated \( M_r \)-distribution. In this case a new \( M_r \)-distribution has to be estimated on the basis of the earlier estimated \( M_r \)-distribution and the calculated \( M_r \)-distribution. The whole process is now started again and will only come to an end if the \( M_r \)-distribution used to calculate the stresses is
almost equal to the Mr-distribution which follows from these stresses.

For this research it was assumed that a 5% accuracy, meaning that the maximum difference between the Mr- and \( \nu \)-values that are used to compute the stresses in the substructure and the Mr- and \( \nu \)-values that follow from these stresses is 5%, is sufficient to obtain a sufficiently accurate stress distribution.

3.3.3 Permanent deformation behaviour

Wheel load stresses in granular materials will cause the development of permanent strain. The first thing that is needed in order to come to a theoretical explanation of rut depth development in concrete block pavements thus is a model which explains the development of permanent strain in granular materials as a function of the stresses to which the material is subjected. If such a model is available then calculated stresses can be translated to permanent strain development throughout the substructure of the block pavement. For this purpose a routine is needed which translates stresses to permanent strains.

Not all wheel loads are applied to the pavement in the exact middle of the wheeltrack. Some wheel loads might be introduced at some distance left of the middle of the track while other wheel loads are introduced right of the wheeltrack. This phenomenon is called lateral wander and results in a certain transversal distribution of wheel loads applied to a pavement (23).

Of course the subroutine translating the calculated stresses into the development of permanent strain should take into account the effects of lateral wander. Given the result of a single resilient calculation this procedure should thus give the development of a rut depth bowl as a function of the number of load repetitions and the amount of lateral wander.

It is then possible to make calculations for any block pavement, loaded by various loads, with numerous contact pressures and showing various amounts of lateral wander. The results of these calculations can be described by a rutting model, explaining the rutting behaviour of a block pavement as a function of the wheel load, the contact pressure, the amount of lateral wander, the materials in the substructure, the thickness of the various layers in the substructure and the properties of the subgrade.

A flow chart of the process which results in a rutting model for concrete block pavements is given in figure 3.4. The process is further explained hereafter.
The process starts with the resilient calculation discussed in paragraph 3.3.2. The result of the resilient calculation is the distribution of the ratio of applied stress over stress at failure of the granular materials in the substructure of the block pavement. The following additional input is needed in order to determine the rut bowl development:

- **Permanent strain development**: For each material in the substructure the development of permanent strain with the number of load repetitions has to be known as a function of the applied stresses, expressed as the ratio of applied stress over stress at failure.

- **Lateral wander**: The shape of the rut bowl of course depends on the amount of lateral wander. As a result the amount of lateral wander has to be taken into account in the determination of the rutting behaviour of block pavements.
On the basis of this additional input the rut bowl development in a block pavement is determined given the distribution of the ratio of stress over stress at failure, this is done in the process "Compute rut depth development". It is now possible to determine the rut bowl development in any concrete block pavement loaded by any load. By repeating the process for numerous block pavement structures, loaded by various loads showing different amounts of lateral wander, a data set is produced, "Rutting behaviour of various block pavements". This data set is analyzed and used for developing a rutting performance model.

The rutting performance model will make permanent deformation predictions a lot easier. Material properties, i.e. the strength, the resilient behaviour, the permanent strain behaviour and the dead weight are no longer needed as input. The model yields insight into the rutting behaviour of a concrete block pavement on the basis of the material(s) used in the substructure. If the joint behaviour is kept constant, as will be done in this research, then the model gives the rutting behaviour of a block pavement on the basis of:

- **Properties of the subgrade:** The properties of the subgrade of course have their effects on the rutting behaviour of a block pavement.
- **Substructure:** The substructure of the block pavement is no longer described by material properties in combination with layer thicknesses. In this stage of the research it is enough to know the layer thicknesses and the type of materials which are applied in the various layers. It is clear that the substructure design has a major influence on the rutting behaviour of a block pavement.
- **Lateral wander:** The amount of lateral wander is also of influence on the rutting behaviour of a block pavement. The rutting performance model thus needs the amount of lateral wander as input.
- **Traffic:** The magnitude of both the wheel loads and the contact pressure affects the rutting behaviour of a pavement and are thus needed as input.

Given the input described above the rutting behaviour of a certain concrete block pavement is determined by the procedure "Determine the rutting behaviour of pavement", see figure 3.5. Now the rutting behaviour of a concrete block pavement is known as a function of the wheel load and the contact pressure. For the determination of the traffic introduced permanent deformation in concrete block pavements further input is needed:

- **Vehicle characteristics:** The traffic that will use the pavement has to be known. Besides the wheel loadings especially the dynamic properties of the various vehicles are of importance. These dynamic properties determine the dynamic effects that will result from initial longitudinal unevenness.
- **Roughness:** Of course the initial roughness of a concrete block pavement has to be known. If a new block pavement shows significant initial
unevenness, large dynamic effects may cause failure on the basis of roughness. If the same pavement does hardly show initial roughness, then there will hardly be dynamic effects. In this case the pavement will probably fail as a result of rutting.

![Flow chart of the determination of the permanent deformation behaviour of concrete block pavements.](image)

Given the vehicle characteristics and the initial roughness, dynamic axle loads are determined by means of vehicle simulations, which are performed in the process "Vehicle simulation". These dynamic axle loads will of course vary with the chainage, so that the rut depth which is caused by these axle loads will also vary over the length of the pavement. This varying rut depth is determined on the basis of the rutting behaviour of the pavement, as calculated in the process "Determine rut depth varying with chainage". A rut depth varying with the chainage of course affects the initial longitudinal profile in the wheeltracks. By considering the effects of the rut depth varying over the length of the pavement a new longitudinal profile is thus obtained. A check is now made, "Check if enough vehicles are simulated". If the number of simulated vehicles equals the desired number of vehicles to be simulated,
then the simulation is stopped and the development of both longitudinal and transversal unevenness is known. If the number of vehicles for which the effects on both longitudinal and transversal unevenness are determined is not enough, then the simulation is continued, using the longitudinal profile in which the effects of a rut depth varying with the chainage are represented.

As will be discussed later in this dissertation the determination of the development of longitudinal unevenness by means of repeated vehicle simulation is straightforward, but very time consuming. Therefore also a more practical model will be developed. This model is however not discussed here.

3.4 Discussion

The procedures and subroutines discussed in the previous paragraphs make it possible to determine the development of traffic induced transversal and longitudinal unevenness. The development of longitudinal unevenness due to other phenomena is not described. Also the development of longitudinal unevenness as a result of the pavement rutting behaviour that might vary over the length of the pavement (as a result of varying layer thicknesses, varying material properties and so on) is not explained.

The procedures described earlier thus consider an ideal concrete block pavement which is homogeneous in its behaviour in the longitudinal as well as the transversal direction. Only as a result of initial longitudinal unevenness and the vehicle responses to this unevenness the development of further longitudinal unevenness is considered.

As described, a lot of complex input is needed in order to be able to develop the rutting performance model which really is the main input for the last flow chart given in figure 3.5. If this model would be available, minor extra procedures would make it into a design method which distinguishes transversal and longitudinal unevenness and thus enables a better concrete block pavement design.

Main assumption

In the previous sections a major assumption or simplification is made. The resilient and permanent deformation behaviour of a concrete block pavement are discussed and considered as two separate types of behaviour. In reality this is not the case. Permanent deformation and resilient deformation both occur in one process and are thus strongly related. In figure 3.6 this is further explained.
Granular materials under cyclic loading collect strain if the load builds up. During unloading the major part of the collected strain proves to be resilient. A minor part however is irreversible, as a result permanent strain will develop. The resilient deformation behaviour and the permanent deformation behaviour of granular materials are thus combined in one process. Separating this single process in two sub-processes thus means that reality, to some extent, is not described correctly.

![Diagram of Load, Deformation, Strain, and Number of Load Cycles](image)

*Fig 3.6* Deformation in granular materials under cyclic loading.

Dividing the single process into two sub-processes can only be done if the permanent strain that is introduced during one load cycle is only a fraction of the total strain introduced during the load cycle. In other words, the behaviour of a pavement can only be considered as being resilient if the permanent strain build up during one load cycle is negligible.

The amount of permanent strain build up for the total pavement structure during one load cycle can easily be estimated. Under a 50 kN load, 0.5 mm to over 2 mm maximum resilient deflections are measured during FWD-measurements. The 15 mm rut depth design life of such a pavement might be 50,000 100 kN standard axle load repetitions (or 50 kN wheel load repetitions). On average one 50 kN wheel load repetition thus introduces an additional 0.003 mm rut depth. The deflection of the pavement under such a load equals about 1 mm, so that the permanent strain build up during one load repetition equals about 0.03% of the total deformation. It might be clear that this small amount of permanent strain is negligible.
Of course a similar consideration can be made on the basis of actual measured material behaviour. For this purpose some results are taken from literature. In the work of Sweere (22) both the resilient behaviour as well as the permanent strain behaviour of amongst others two materials, i.e. a sand (material number so2) and a crushed lava (material number co2) are discussed. Based on regression models Sweere fitted through the triaxial test data the following plots were back-calculated.

**fig 3.7** Development of back-calculated axial strain for a crushed lava (22) during the first 20 load cycles.

**fig 3.8** Development of back-calculated axial strain for a sand (22) during the first 20 load cycles.

Sweere expressed the cyclic stress applied to a sample as the ratio of applied cyclic stress over applied confining stress "\(\sigma_c/\sigma_3\)". Figure 3.7 shows...
that the permanent strain build up in the crushed lava is only limited compared to the resilient strains that develop during loading. For the sand, figure 3.8, it was found that the permanent strain build up during the first load repetitions is quite large. With an increase of the number of applied load repetitions the build up of permanent strain however strongly reduces. This is shown in figure 3.9 which gives the development of the ratio of permanent strain over resilient strain for the first 50 load cycles.

![Diagram](image)

*fig 3.9* Development of back-calculated ratio of permanent strain over resilient strain during the first 50 load cycles.

When a ratio of permanent strain over resilient strain "ε_p/ε_r-ratio" of 5% is considered to be a small enough value to sub-divide the single process of strain development, figure 3.9 shows that only during the first load repetitions sub-dividing might result in some errors. The number of load repetitions for which an error might be made varies from 0 to about 45, depending on both the material type and the magnitude of the cyclic load to which the material is subjected.

Since concrete block pavements mostly have a design life in excess of 50,000 standard load repetitions it is clear that the error that might be a result of the sub-dividing of the single strain process can only be limited.

### 3.5 Objectives of this research program

The objectives of this study are to explain the traffic induced permanent deformation behaviour of concrete block pavements as accurate as possible on the basis of measured material behaviour and theoretical pavement structure analyses. This approach will make it possible to determine the usability of
new road building materials in block pavements without the extensive in-practise testing by means of test sections. Finally it will deliver possibilities to optimize the designs for this pavement type. This method should make a distinction between the two types of most common block pavement damage, namely transversal unevenness or rutting and longitudinal unevenness or roughness. The design method should be both a verification and an extension of the C.R.O.W D3 method in which only rutting is taken as the design criterion.

3.6 Approach of the research program

Hereafter the approach chosen for this research program is discussed. The program roughly consists of two parts: a data collecting part and a theoretical part, see figure 3.10. The theoretical part has been discussed in the previous sections. It was shown that a lot of complex input is needed in order to determine the behaviour of a concrete block pavement. The various input values will be determined on the basis of the results of the data collection which was done as part of this research.

![Flow chart of the research performed to reach the objectives.](image)

fig 3.10 Flow chart of the research performed to reach the objectives.
The data collection starts with Falling Weight Deflection (FWD) measurements and longitudinal and transversal unevenness measurements on seven in-service concrete block pavements. The results from these measurements gave information of the condition of these concrete block pavements.

The FWD-results are used to obtain information about the stiffness of the joints between the concrete blocks, see chapter 5.

Samples of the granular materials taken from the substructure of the seven pavements were extensively tested in the Road and Railroad Research Laboratory of the Delft University of Technology. The behaviour of the granular materials as measured in the laboratory, has been described by material models. These models were implemented in the theoretical part of this research. Hereafter rut depth calculations were performed on the basis of the material behaviour measured in the laboratory. As discussed earlier the results of these calculations were used to develop an accurate description of the rutting performance of concrete block pavements.

The next step was to simulate the development of traffic induced longitudinal unevenness in concrete block pavements. Here to initial longitudinal profiles were needed. Such profiles were determined on the basis of the profile measurements performed on the in-service block pavements.
4

A structural model for the resilient analysis of Concrete Block Pavements

4.1 INTRODUCTION

In this chapter the development of a structural model for the resilient analysis of concrete block pavements is discussed. Special attention is paid to the modelling of the discontinuous concrete block layer.

Since the granular materials used in the substructure of a concrete block pavement show stress-dependent resilient behaviour, the structural model should be developed in such a way that this material behaviour can be implemented as well.

4.2 HISTORICAL REVIEW

The discontinuous character of a concrete block layer forms a major problem in modelling concrete block pavements. This discontinuous block layer cannot be represented in a multi-layer program. For this reason the Delft University of Technology started modelling concrete block pavements by means of Finite Element Models in the early 1980's.

The first structural model for concrete block pavements, developed by Kok, was two dimensional (7, 14, 24, 25, 26). The concrete blocks were represented by rigid bodies, see figure 4.1. These rigid bodies were interconnected by means of shear springs. Between the modelled block layer and the representation of the substructure a layer of springs was projected. This layer of vertical springs represented the bedding sand layer.

The described representation of a concrete block pavement had a few major disadvantages. The most important disadvantage of this model was that its two dimensional character implied, that pavements could only be analyzed in plain strain or plain stress conditions which in reality do not occur in pavements. Another important disadvantage of this model was the poor representation of the bedding layer by means of springs.
A second model was developed by Jacobs and Kok in 1987. This model did not have the major shortcomings of the first model. The 1987 model was axial symmetric and again had a representation of the blocks formed by rigid bodies \((7, 27, 28)\). As a result of this block representation (rigid bodies in an axial symmetric model) the modelled blocks could not rotate. Due to the inability of the blocks to rotate, large vertical tensile stresses would develop in the bedding sand layer, see figure 4.2. The development of these tensile stresses is not realistic since sand cannot take tensile stresses.

To prevent the development of these tensile stresses special attention was paid to the connection between the modelled blocks and the substructure underneath the blocks. It was chosen to connect the blocks with the substructure only at one node, see figure 4.3.
The main shortcomings of the 1987 model had to do with the representation of the blocks by rigid elements. In an axial symmetric model each element really is a ring around the axis of symmetry. Concrete blocks modelled by rigid bodies can as a result not rotate or translate in the horizontal direction. The blocks in the 1987 model could thus only translate in the vertical direction. This implies that only shear loads in the joints can develop, which explains why the joints are modelled by shear springs only.

Since the concrete blocks in the 1987 model could not rotate, joint compression was not modelled. The development of compressive joint forces acting to expand the block layer was thus not modelled. As a result the model did not indicate the need for edge restraints.

![Diagram](image)

*fig 4.3* The 1987 model developed by Jacobs and Kok.

![Diagram](image)

*fig 4.4* The 1989 model developed by the author and Kok.

In 1989 the author and Kok developed a purely three dimensional model for the resilient analysis of concrete block pavements (18, 19). Again the
concrete blocks were represented using rigid bodies and interconnected by springs. In this model, however, shear springs as well as normal springs were needed to represent the joints, see figure 4.4.

Given the fact that the 1989 model was a 3-D model, the blocks were able to rotate around any axis and translate in any direction. As a result of block rotation the blocks in this model were able to smoothly follow the deflection bowl of the bedding sand layer, so that only minor vertical tensile stresses were computed in the bedding sand layer.

These minor tensile stresses are a result of the compressive normal joint forces that developed as a result of block rotations. These compressive normal joint forces of course somewhat limited the block rotations so that minor tensile stresses were still introduced, see also figure 4.2.

In the 1989 model the representation of the block layer was very close to reality. The blocks were modelled in herringbone bond, the bond type which is used for most concrete block pavements. The equivalent stiffness of the block layer was not formed in any way by the properties of the individual blocks, but was completely dependent on the development of inter block reactions (joint forces). The results obtained by this model showed that concrete blocks in a block layer are subjected to translations as well as rotations when the pavement is loaded. As a result of these rotations and translations compressive joint normal forces will develop in the concrete block layer.

Despite the good representation of the concrete block layer, the 1989 model also had a few disadvantages. The main disadvantage was that the analysis of a true pavement resulted in an excessive computation time. The reasons for this are that in a true 3-D model each node results in three degrees of freedom while a node in an axial symmetric model only results in two degrees of freedom. Furthermore a true 3-D model requires far more nodes than an axial symmetric model.

Because of these limitations the 1989 model only had 5 element layers, used to represent the man-made pavement structure. The subgrade was represented by means of vertical springs. In total the model consisted of 27 concrete blocks, so only an area of 0.6 m² was modelled.

Another disadvantage of the 1989 model was its very complex geometry. A lot of imagination was needed to visualise the model and its calculation results.
A large disadvantage of all the models developed in the 80's is that the stress-dependent resilient behaviour of the granular materials in the substructure of concrete block pavements was not implemented.

4.3 A NEW AXIAL SYMMETRIC MODEL FOR THE RESILIENT ANALYSIS OF CONCRETE BLOCK PAVEMENTS

As discussed in the previous section, the finite element models developed at the Delft University of Technology in the 1980's all had their shortcomings. These models were either too small in size or they had a poor representation of the concrete block layer. For this research a new structural model was therefore needed.

This new model had to combine the good properties of the previous models in one model, without showing the shortcomings of these earlier models. This means that the model should be able to represent a pavement structure in a realistic way, using only limited degrees of freedom. The representation of the block layer should allow block rotation and translation and as a result it should be able to show the development of normal joint forces. Furthermore the geometry of the model should not be too complicated.

Limitation of the amount of degrees of freedom is very important. The granular materials applied in the substructure of a concrete block pavement show stress-dependent resilient behaviour. This behaviour had to be taken into account, which means that a single calculation becomes an iterative process, as is discussed elsewhere in this dissertation.

In order to limit the degrees of freedom of the model on one hand and to be able to model a large volume on the other, it was decided to make an axial symmetric model for the resilient analyses of concrete block pavements. This of course meant that the block layer could not be modelled as was done in the 1989 model.

Special attention was therefore paid to the representation of the concrete block layer in this axial symmetric model. This layer should be able to react as expected from reality and as computed by the 1989 model, see figure 4.5.

In figure 4.5 a small part of a concrete block layer is presented under both unloaded and loaded conditions. As shown by this figure block rotation and translation occurs. The blocks themselves do not have any resistance against these rotations and translations, joint forces will however develop due to the block movements. As a result of these inter block joint forces the concrete block layer as a whole obtains a certain (equivalent) stiffness.
Fig 4.5  Block rotation and translation due to a (wheel) load.

The elements in an axial symmetric model are really rings around the central model axis. Concrete blocks modelled by these elements thus show an enormous resistance against rotation and horizontal translation. Horizontal translation results in ring diameter change while block rotation means ring shape deformation, see figure 4.6.

Fig 4.6  Stress free ring deformation needed for a good axial symmetric block layer representation.

It is however possible to create axial symmetric elements that can undergo stress free ring diameter change and ring shape deformation. In normal axial symmetric elements these deformations result in stresses in the $O$-direction, $o_{oo}$. By modifying the properties of the elements used to represent the blocks in such a way that they can no longer build up stresses in

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the θ-direction, a better axial symmetric representation of the block layer is obtained, i.e. blocks can now rotate and translate freely. As a result the modelled block layer will no longer show any self-confining properties, joint normal forces will develop that act to expand the block layer. Like in reality the stiffness of the block layer now completely depends on inter block reactions.

Since the concrete blocks in the model can now rotate, no tensile vertical stresses are to be expected in the bedding layer. As a result the blocks can be placed over the bedding sand layer in a straight forward way, as was done in the 3-D 1989 model, see figure 4.7.

![Diagram](image)

**fig 4.7** Upper part of the new structural model.

Alike in the models discussed earlier, the springs representing the joints are dimensionless. This is necessary since the nodes of the elements representing the blocks can only be subjected to forces and not to moments. The blocks in the new model are 160 mm wide and have a thickness of 80 mm. This thickness equals the thickness of the most commonly used concrete blocks for roads in the Netherlands. While 160 mm equals the average of the horizontal dimensions of these blocks (211 x 105 mm²) taking into account a 2 mm wide joint, ([211+2+105+2]/2=160). The width of the joints is thus accounted for in the dimensions of the concrete blocks.

In figure 4.8 the basic geometry of the complete model is presented. In the r-direction the elements have the following dimensions: 8 x 32 mm, 4 x 80 mm, 4 x 160 mm, 5 x 320 mm, 2 x 640 mm and 1 x 1280 mm. The radius of the model thus becomes 5,696 mm. The first concrete block has a
radius of 96 mm and is formed by the first 3 elements. The following seven concrete blocks are 160 mm wide. At greater distances the dimensions of the blocks further increase.

\[ \text{fig 4.8} \quad \text{Basic geometry of the axial symmetric model.} \]
The dimensions of the elements in the z-direction, plotted in figure 4.8, are: 1 x 80 mm, 2 x 25 mm, 7 x 50 mm, 2 x 100 mm, 3 x 200 mm, 1 x 300 mm, 3 x 400 mm, 2 x 800 mm, 2 x 1600 mm, 1 x 3200 mm and 1 x 6400 mm, giving the model a depth of 17,180 mm.

The nodes at the bottom of the model can not move in any direction, whereas the nodes at the edge of the model can only move in the z-direction. Given the large dimensions of the model these boundary conditions hardly effect the results that are obtained in the upper central part of the model, which is the part where all interest is focused on.

As an example figure 4.9 gives the element mesh in relation to a particular pavement structure. In this figure only the upper part of the mesh is presented. Within this mesh the layers of a concrete block pavement with a 1500 mm thick substructure are indicated; the base course has a thickness of 250 mm.

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**fig 4.9**  Upper part of the axial symmetric model.

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The mesh as discussed here is the basic mesh as used for the calculations in this research program. Minor modifications of this mesh were sometimes needed in order to bring it in agreement with the substructure design of a pavement that was to be analyzed. These minor modifications are not discussed.
4.4 CHARACTERISTICS OF THE BLOCK LAYER REPRESENTATION

As discussed in the previous sections the characteristics of the block layer are very important and in this section these characteristics are discussed. Use is made of two linear calculations. In one calculation, "calculation A", the elements in the blocks are "normal" axial symmetric elements. This implies that the concrete blocks can not rotate end translate freely. In the other calculation, "calculation B", the elements in the blocks cannot build up stresses in the Θ-direction (except for the central block which is again modelled by "normal" elements). As a result the modelled blocks can now translate and rotate freely as is the case in reality.

It is expected that "calculation A" will result in tensile stresses in the bedding layer. Within "calculation A" the block layer strongly resembles the block layer as modelled in the 1987 model.

Within "calculation B" these problems are no longer expected. The modified elements used in the block layer are expected to react similar to the concrete blocks within the 1989 3D model.

In order to prove these expectations a pavement structure was analyzed. This pavement structure consisted of a 80 mm thick concrete block layer, placed over a 50 mm bedding sand layer. Underneath the bedding layer a 200 mm base course was placed over a 750 mm sand sub-base. Both the sands as well as the base material had a Poisson's ratio, ν, of 0.35. The modulus of elasticity, E, of the sands was chosen to be 100 MPa while for the base material an E-value of 300 MPa was used. The subgrade had an E-value of 60 MPa and a ν-value of 0.45.

The joint normal spring stiffness was 550 N/mm per mm modelled joint length and the joint shear spring stiffness was 50 N/mm/mm. Both springs were inactive in case of a tensile normal joint force. As a result of these weak joints, see chapter 5 for the determination of the joint stiffnesses, the behaviour of the modelled block layer mainly depends on the properties of the block representation.

The load applied to the pavement was a 50 kN wheel load with a circular contact area with a radius of 150 mm.

Figure 4.10 shows the effects of the described modification of the elements used for modelling the concrete blocks on the computed deflection bowls. As shown in this figure the deflection bowl based on normal elements "calculation A" resembles a staircase. A much smoother deflection bowl is obtained when the blocks are given the freedom of rotation and translation.
Fig 4.10 Deflection bowls computed for two block layer representations.

The shape of a deflection bowl is of major importance on the stresses that develop directly underneath the blocks. In figure 4.11 it is shown that fairly large tensile vertical stresses, $\sigma_{zz}$, are computed at a depth of 92.5 mm, i.e. 12.5 mm below the blocks, in the case the blocks are modelled by normal elements. By comparing figure 4.11 with figure 4.10 it can be seen that these tensile stresses are a result of the limited block rotations.

Fig 4.11 Vertical stress, $\sigma_{zz}$, in the bedding sand directly underneath the block layer.
By allowing the blocks to rotate large tensile stresses no longer develop. As shown by figure 4.11 a minor tensile vertical stress is still computed underneath the fourth concrete block. This tensile stress is a result of inter block reactions (especially compressive normal joint forces) that somewhat limit block rotation.

The block layer modeled using "normal" elements, "calculation A", shows major self-confining properties. When the "modified" elements are used to model the blocks, the freedom of horizontal translation results in a toplayer without any self-confining properties. It is clear that this difference between the two block layer representations effects the development of joint normal forces, see figure 4.12.

![Joint normal forces for the two types of block representations.](image)

As shown in figure 4.12 there is an enormous difference in the computed compressive normal joint forces that develop in the block layers. In case the concrete blocks can not rotate and translate freely only limited joint normal forces are computed. This is a result of the limited block rotations and the self-confining properties of the block layer.

When the concrete blocks can rotate and translate freely, "calculation B", far greater joint normal forces are computed as a result of block rotation. Since the block layer no longer shows self-confining properties, the need for an edge restraint is now shown by the model.
For the joint shear forces a completely different picture is expected. The minor block rotations that occur in "calculation A" result in large vertical displacements over a joint, see figure 4.10. These large vertical displacements over a joint of course result in large joint shear forces. The development of these shear forces in relation to the distance from the load centre is plotted in figure 4.13.

Fig 4.13 Joint shear forces for the two types of block representations.

The concrete block layer transduces the wheel load to the underlying substructure and thus affects the stresses and deformations that will develop in the substructure. Figures 4.14 and 4.15 give insight into the effects of the block layer properties on the deformations in the substructure. In these figures the deformations of a small part of the finite element mesh are presented for both calculations. In both these figures the mesh deformation is magnified by a factor of 100.

In figure 4.14 the mesh deformation over a radius of 1056 mm and a depth of 680 mm for "calculation A" is given. In this calculation the concrete blocks were modelled using "normal" elements. In the figure it is clearly shown that this representation results in a very irregular mesh deformation in the bedding layer. As mentioned before, these deformations of course largely explain the development of tensile vertical stresses, $\sigma_{zz}$, in the bedding layer, see figure 4.11.

When the concrete blocks are given the freedom of rotation and translation, "calculation B", a completely different mesh deformation is obtained. In this case the deformations in the bedding layer are much
smoother. This is a direct result of the ability of the blocks to rotate and translate resulting in a smooth deflection bowl.

**Fig 4.14**  Mesh deformation obtained by "calculation A".

**Fig 4.15**  Mesh deformation obtained by "calculation B".
In the figures 4.14 and 4.15 the concrete blocks in the deformed mesh scam to overlap each other. This is a result of the modelling of the joints. The springs within these joints are dimensionless since nodes cannot be subjected to moments that would occur if the shear springs would have a certain length. This means that joint compression results in the blocks overlapping each other. Given the magnification of the deformations by a factor of 100, the actual block overlap is only 1% of the overlap shown in the figures 4.14 and 4.15.

As stated earlier the dimensions of the modelled blocks incorporate a 2 mm wide joint. This means that as long as the joint compression is less than 2 mm no direct contact between blocks is computed.

Given the compressive normal joint spring stiffness of 550 N/mm/mm, a 2 mm joint compression would result in a 1100 N/mm normal joint force. In figure 4.12 it is shown that the maximum normal joint force equals almost 55 N/mm, so that the maximal joint compression equals almost 0.1 mm. Given the incorporated joint width of 2 mm this implies that no direct contact between blocks occurs.

4.5 VALIDATION OF THE PROGRAM CODE

In order to ensure that the computer code of the finite element structural model did not contain errors it was validated by comparing its results with those obtained by the well known linear-elastic multi-layer program BISAR (29). BISAR is taken as the reference since it is the best known program in the pavement engineering world, and is generally accepted as a program giving reliable calculation results.

Since BISAR is a multi-layer program it cannot handle a discontinuous top-layer and therefore a pavement with a continuous top-layer was analyzed. This top-layer had a thickness of 80 mm, its Young's modulus was 3000 MPa and its Poisson's ratio 0.25, which are reasonable values for representing a concrete block layer in a multi-layer program (9, 26, 30).

In the finite element model the top-layer was modelled as a continuous layer without joints. Of course no use was made of the modified elements as applied for the blocks in "calculation B", discussed in the previous paragraph.

Underneath this top-layer a 50 mm thick bedding sand layer with a Young's modulus of 100 MPa was projected which on its turn was placed on top of a 200 mm thick base layer with a Young's modulus of 300 MPa. The last layer, representing the subgrade, had a Young's modulus of 100 MPa. The three layers below the top-layer all have a Poisson's ratio of 0.35.
Since the mesh used for finite element calculations has its effects on the computation results, the mesh used for this comparison largely equals the mesh discussed earlier. The only difference is that the top-layer is modelled by means of two element layers, both with a height of 40 mm.

The load applied to the four layer structure is a 50 kN load with a radius of 150 mm.

The first comparison refers to the computed deformation. It is should be noted that a Finite Element Model, FEM, is based on the determination of displacements. On the basis of these displacements, stresses and strains can be computed.

In figure 4.16 the deflection bowls computed by both BISAR and the FEM are presented. The figure shows that the FEM computes slightly larger deflections than BISAR does, if the distance to the load centre becomes larger than 3000 mm. This is caused by the finite dimensions of the Finite Element Model. The edge of the mesh was 5696 mm from the load centre and the nodes there were allowed to displace freely in the vertical direction.

![Deflection bowl computed by the FEM and BISAR.](image)

Fig 4.16  Deflection bowl computed by the FEM and BISAR.

By plotting the deflections against the depth it is shown that the FEM computes smaller deflections than BISAR if the depth becomes larger than about 10,000 mm, see figure 4.17. Again this is a result of the finite dimensions of the FEM. At a depth of 17,180 mm the deflections in the FEM computation equal zero since the bottom of the model is located at this depth. At the bottom of the FEM the nodes cannot move in any direction. The BISAR model of course represents an infinite depth and thus calculates minor deflections at this depth. As is shown by figure 4.18 however the deflections
in the upper part of the FEM agree very well with the values calculated with BISAR.

**fig 4.17** Deflections plotted against the depth.

**fig 4.18** Deflections, in the upper part of the pavement, plotted against the depth.

The figures 4.16, 4.17 and 4.18 show that the deformations computed by the FEM closely resemble the deformations computed by BISAR. Since the stresses are computed on the basis of these deformations, it is expected that the same will hold for the computed stresses.

The normal stresses calculated by both methods, at $r=16$ mm, are

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plotted against the depth in figure 4.19. The figure shows that both BISAR and the FEM calculate large horizontal stresses in the top-layer. Figure 4.20 gives a detailed picture of these large stresses calculated for the top-layer. As shown in this figure the agreement between BISAR and the FEM is so good that the lines for both models can hardly be distinguished.

**fig 4.19**  Normal stresses against the depth (r = 16 mm).

**fig 4.20**  Normal stresses in the top-layer (r = 16 mm).

Figure 4.20 shows that the top-layer acts as a slab (which it is). In the centre of the model horizontal compressive stresses are computed in the upper part of this layer. Tensile horizontal stresses develop in the lower part of this layer.
Figure 4.21 gives a detailed plot of the stresses that are found at minor depth below the top-layer. In this plot the relatively stiff base layer can easily be recognized.

![Stress vs Depth Graph](image)

**fig 4.21** Normal stresses against the depth \( (r = 16 \text{ mm}) \).

Figures 4.22 shows the normal stresses, in relation to the horizontal distance to the load centre, in the upper part of the bedding sand layer \( (z = 92.5 \text{ mm}) \). Figure 4.23 is the enlargement of the most important part of figure 4.22.

![Stress vs Horizontal Distance Graph](image)

**fig 4.23** Normal stresses in relation to the horizontal distance to the load centre \( (z = 92.5 \text{ mm}) \).
Similar to figure 4.22, figure 4.24 gives the stresses that develop 25 mm above the bottom of the base layer ($z=305$ mm). Figure 4.25 is an enlargement of the interesting part of figure 4.24.
The figures 4.16 to 4.25 show that the FEM program code leads to results that closely resemble the results obtained by the BISAR program. The agreement between the two models is often so strong that the lines obtained with BISAR and the lines obtained with the FEM overlap. Minor differences are however present. These differences are mainly caused by the fact that it is not possible to create completely equal models with both program codes. The differences caused by the difference in size of the models (infinite against finite) are mainly found at larger distances from the load centre (large z-coordinate and/or large r-coordinate), i.e. at the edges of the FEM.

Of course the area near the load is the most interesting part of both the models. Within this area the largest stresses and strains are found. The errors that are a result of the difference in size of the two models are very small within this interesting part, implying that the size of the FEM is large enough to limit the effects of the model boundaries, i.e. the boundary conditions.

Taking into account the differences between the FEM-approach and the multi-layer-approach this section clearly shows that the FEM computer code is correct. Given the small differences between BISAR and the FEM it is furthermore concluded that the mesh presented in the figures 4.8 and 4.9 is fine enough to come to reliable results.
4.6 CONCLUSIONS

The structural model for the resilient analysis of concrete block pavements as described in the previous section is expected to be a powerful tool in the further analysis of concrete block pavement behaviour. Of course this model is still axial symmetric and thus not capable in representing a block layer in herringbone bond. However, using the modified elements to model the concrete blocks, the disadvantages of axial symmetric modelling of concrete block pavements are largely overcome.

A large advantage of the axial symmetric approach is that it contains a limited amount of degrees of freedom. Stress-dependent material behaviour can thus be implemented without the danger of unacceptable long computing times. Another advantage of the model as presented, is that it shows a simple geometry. The results of the computations can thus be interpreted quite easily.
5 Stiffness of the joints

5.1 INTRODUCTION

As discussed earlier, it was decided to explain the development of traffic induced permanent surface deformation in concrete block pavements in a theoretical way. As was shown in chapter 3 this approach amongst others implies that the behaviour of the joints between the blocks has to be known.

For the determination of the joint behaviour of in-service concrete block pavements no laboratory tests were done. It was decided to determine the joint stiffness on the basis of in-situ Falling Weight Deflection measurements (FWD-measurements).

In this chapter the seven test sections, where these FWD-measurements were performed, are described first of all. Hereafter the FWD-measurements are discussed. First of all the Falling Weight Deflectometer is discussed, followed by a description of the FWD-data processing. Then the FWD-results are presented and shortly discussed.

Then it is explained in what way the joint stiffnesses are determined using the FWD-data. By discussing the FWD-results obtained for two test sections it is then shown that the determined joint stiffnesses are in agreement with the measurements.

5.2 TEST SECTIONS

5.2.1 Introduction

For this research measurements were performed on seven in-service test sections. Furthermore samples of the road building materials used in the substructure of these seven test sections were taken for laboratory testing, see chapter 6. These seven test sections were chosen from an initial set of 31 sections. On the initial 31 test sections both profile measurements (longitudinal and transversal) and FWD-measurements were performed. On the basis of the results of these initial measurements the seven test sections were selected.
The selection of the seven test sections was mainly based on two arguments. First of all the sections had to be structural homogeneous over the length. FWD-data were used to determine if this was the case.

The model described in chapter 4 requires joint stiffnesses that are representative for Dutch concrete block pavements. As was shown by the results of the C.R.O.W working group D3, the resilient behaviour of a concrete block pavement can change during trafficking (progressive stiffening). It is possible that a part of this progressive stiffening is caused by changes in the joint behaviour. To obtain representative values for the joint spring stiffnesses it was therefore decided to select concrete block pavements that showed a strong variation in their residual design life and substructure design.

5.2.2 Description of the seven in-service test sections

5.2.2.1 General

In this paragraph a short description of the seven test sections is given. As will be shown, four of these sections are located in the municipality of Zaanstad. The subgrade in that municipality consists of a 5 m thick peat layer with a low bearing capacity. This peat layer is situated on top of a sand layer with a thickness of about 1 m. Underneath this thin sand layer another 10 m thick peat layer is present. In the municipality of Zaanstad very high groundwater levels are found.

The other three test sections are located in the southern part of the city of Rotterdam. The subgrade in this part of Rotterdam varies from clay to sandy clay.

All sections have a length of somewhat more than 102 m.

In the following, a brief description of the pavement structures of the seven test sections is presented. It is stated that the layer thicknesses were not measured. The mentioned information on the substructure designs was given by the local road authorities.

For all test sections also an indication of the traffic that is using the pavements is given. These figures are all based on estimations made by the author on the basis of information given by the road authorities combined with impressions of the actual traffic. No traffic counts or axle load measurements were performed. In the case that a pavement is part of a bus route, then the number of public transport busses that travel over the pavement is given. This information is based on the time schedules of the bus companies that operate the particular bus lines.
To give an impression of the structural condition of the pavements some figures about rutting are given. For each pavement the rut depth underneath a 1.2 m straight edge or relative rut depth, RD_r, with a 30% probability of exceeding, is given. For the sake of completeness it is noted that RD_r,30% is calculated using the mean and standard deviation of RD_r. The mentioned figures refer to the average value of RD_r,30% as determined for the left and right wheel track.

The mentioned rut depth was measured in July 1991 for the pavements in Rotterdam and August 1991 for the pavements in Zaanstad.

5.2.2.2 Sections in Rotterdam

Baarsweg
- 90 mm concrete blocks in herringbone bond,
- about 900 mm thick sand sub-base.

The pavement was constructed and opened for traffic in the year 1986. The Baarsweg is a quiet, not so heavily trafficked street in a residential area. The Baarsweg is trafficked by 13,364 busses per year per direction. It is estimated that about 10,000 commercial vehicles per year per direction travel over the Baarsweg. The average RD_r,30% in July 1991 was about 6 mm.

M. Havelaarweg
- 80 mm concrete blocks in herringbone bond,
- 50 mm bedding sand layer,
- 300 mm crushed masonry/concrete base layer,
- 600 mm sand sub-base.

This pavement was completed in the year 1990. The M. Havelaarweg is trafficked by 33,540 busses per year per direction. It is estimated that about 50,000 commercial vehicles pass the M. Havelaarweg per year per direction. The average RD_r,30% was about 5 mm in July 1991.

Pascalweg
- 80 mm concrete blocks in herringbone bond,
- 50 mm bedding sand layer,
- 300 mm crushed masonry/concrete base layer,
- about 600 mm sand sub-base.
The Pascalweg was constructed in the year 1990. It is a heavily trafficked main road in a residential area that is travelled by 46,800 busses per year per direction. It is estimated that about 100,000 commercial vehicles travel over the pavement per year per direction. The average $\text{RD}_{r,30\%}$ in July 1991 was about 6 mm.

5.2.2.3 Sections in Zaanstad

**Allanstraat**
- 80 mm concrete blocks in herringbone bond,
- 50 mm bedding sand layer,
- 250 mm crushed masonry/concrete base layer,
- about 300 mm sand sub-base.

The groundwater level is about 600 mm underneath the pavement surface. The pavement structure was constructed in 1991 and opened for traffic in the beginning of September of that year. The Allanstraat is of importance for local traffic and is substantially trafficked. Each year 9,412 busses per direction travel over the Allanstraat. About 25,000 commercial vehicles pass the Allanstraat per year per direction. In August 1991 the average $\text{RD}_{r,30\%}$ was about 4 mm.

**C. Bruynweg**
- 80 mm concrete blocks in herringbone bond,
- 50 mm bedding sand layer,
- 250 mm crushed masonry/concrete base layer,
- about 600 mm sand sub-base.

This pavement was constructed and opened for traffic in the year 1988. The C. Bruynweg is a quiet street in a residential area. It is trafficked by 16,970 busses per year per direction. About 15,000 commercial vehicles travel the C. Bruynweg per year per direction. In August 1991 the average value of $\text{RD}_{r,30\%}$ was about 6 mm.

**Weiver**
- 70 mm concrete blocks in herringbone bond,
- 600 to 800 mm sand sub-base.

This pavement was constructed and opened for traffic in the year 1981. The pavement is located close to the Allanstraat, again the groundwater level
is about 600 mm below the pavement surface. The Weiver is a very long
dead-end street with a length of several kilometres. The test section is located
at the beginning of this street. It is estimated that about 5,000 commercial
vehicles travel the test section per year per direction. The average $RD_{r,30\%}$ in
August 1991 was about 26 mm.

Zaanweg
- 80 mm concrete blocks in herringbone bond,
- about 900 mm sand sub-base.

This pavement was constructed in the year 1988. The Zaanweg is a
heavily trafficked street in a shopping area. It is trafficked by 16,970 busses
per year per direction and it is estimated that about 130,000 commercial
vehicles travel the Zaanweg per direction per year. In August 1991 the
average $RD_{r,30\%}$ was about 8 mm.

5.3 FALLING WEIGHT DEFLECTION
MEASUREMENTS

5.3.1 Introduction

Falling Weight Deflectometers are widely used for nondestructive
pavement testing. By dropping a weight, the apparatus is capable in applying
a pulse-load to the pavement during a short period of time (about 0.025 s). In
most of the cases the magnitude of the load is 50 kN, applied through a
circular foot plate with a 150 mm radius, see figure 5.1. By changing the
magnitude of the falling weight or the falling height, the load applied to the
pavement can be adjusted to the desired level.

During a FWD-measurement the load signal and the signals from the
deflection sensors are constantly sampled. The maximum values of these
signals are displayed or stored in a computer. By plotting the measured
maximum deflections against the distance from the load centre the deflection
bowl due to the applied load is obtained, see figure 5.1.

In the above only a very short description of FWD-measurements and
the used apparatus is presented, the equipment and the measurements are
discussed in more detail elsewhere (44).
For this research program a Falling Weight Deflectometer equipped with six deflection sensors was used. This Falling Weight Deflectometer was built by the Delft University of Technology and is equipped with Dynatest 7800 electronics (44). For this research the deflection sensors were placed at 0 m, 0.3 m, 0.6 m, 0.9 m, 1.2 m and 1.8 m from the centre of the foot plate, as indicated in figure 5.1. The combination of the magnitude of the mass that is dropped (falling weight) and the height from which it is dropped (falling height) is chosen such that the applied load becomes about 50 kN.

As mentioned, all the test sections have a length of somewhat more than 102 m. On each section 11 measurement points (10 m separated from each other) in the right wheeltrack and 11 points between the wheeltracks are used for the measurements.

5.3.2 Data processing

During the measurements, the actual applied force varies somewhat around the target value of 50 kN. This implies that the 22 FWD-bowls measured on one section might all be a result of a somewhat different maximum load value. To make the 22 individual FWD-bowls comparable the first step in the data processing is the correction for the load magnitude. Hereto all measured deflections are linearly corrected to the deflections due to a 50 kN load.
The next step is the computation of the IDK$_{0.3}$ (14). As shown by equation 5.1 the value of IDK$_{0.3}$ is determined by the deflections in the load centre and the deflection at a distance of 0.3 m. The value of the IDK$_{0.3}$ is an indicator of the stiffness of the toplayer. Since this research is concerning concrete block pavements the value of IDK$_{0.3}$ thus gives an indication of the joint behaviour,

$$IDK_{0.3} = d_0 - d_{0.3} \quad (5.1)$$

where:

- IDK$_{0.3}$: surface curvature index [µm]
- $d_0$: deflection at load centre due to a 50 kN standard load [µm]
- $d_{0.3}$: deflection at a distance of 0.3 m from the load centre due to a 50 kN standard load [µm]

After the IDK$_{0.3}$ is determined for all individual measured deflection bowls the last step in the data processing is performed. This last step is to determine the average values of the corrected deflections and the IDK$_{0.3}$. Also the coefficient of variation of these values is determined. Within this last step of the data processing a strict distinction is made between the values measured in the right wheeltrack and the values measured between the wheeltracks.

The determined average deflections and IDK$_{0.3}$ values combined with their coefficients of variation are from now on considered as being good representative values of the FWD-data. In the next section the most important FWD-parameters as measured on the test sections are briefly discussed.

5.3.3 Falling Weight Deflection measurement results

As stated earlier the goal of the FWD-measurements is to obtain proper representative stiffnesses for the springs representing the joints between the concrete blocks. To diminish the variation of the joint stiffness over the seasons, if any, it was decided to perform repeated FWD-measurements. These repeated measurements should cover a complete year. Apart from the initial measurements, 13 additional measurements were therefore planned.

On six of the seven test sections in total 14 series of FWD-measurements were performed. On the Wever only 13 series of measurements could be made. This section was under reconstruction when the last series of FWD-measurements was performed, see table 5.1.
<table>
<thead>
<tr>
<th>FWD-measurement series</th>
<th>Date</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Zaanstad</td>
<td>Rotterdam</td>
</tr>
<tr>
<td>1</td>
<td>16-11-1992(^{1})</td>
<td>03-12-1992(^{2})</td>
</tr>
<tr>
<td></td>
<td>17-11-1992(^{2})</td>
<td>17-11-1992(^{2})</td>
</tr>
<tr>
<td>2</td>
<td>30-09-1993</td>
<td>20-09-1993</td>
</tr>
<tr>
<td>3</td>
<td>21-10-1993</td>
<td>20-10-1993</td>
</tr>
<tr>
<td>4</td>
<td>18-11-1993</td>
<td>15-11-1993</td>
</tr>
<tr>
<td>5</td>
<td>14-12-1993</td>
<td>15-12-1993</td>
</tr>
<tr>
<td>6</td>
<td>13-01-1994</td>
<td>10-01-1993</td>
</tr>
<tr>
<td>7</td>
<td>08-02-1994</td>
<td>07-02-1993</td>
</tr>
<tr>
<td>8</td>
<td>08-03-1994</td>
<td>07-03-1994</td>
</tr>
<tr>
<td>9</td>
<td>06-04-1994</td>
<td>05-04-1994</td>
</tr>
<tr>
<td>10</td>
<td>29-04-1994</td>
<td>03-05-1994</td>
</tr>
<tr>
<td>12</td>
<td>05-07-1994</td>
<td>04-07-1994</td>
</tr>
<tr>
<td>14</td>
<td>08-09-1994(^{5})</td>
<td>31-08-1994</td>
</tr>
</tbody>
</table>

**Table 5.1** Dates at which the FWD-measurements were performed.

\(^{1}\): first measurements on the Zaanweg and the C. Bruynweg,
\(^{2}\): first measurements on the Allanstraat and the Weiver,
\(^{3}\): first measurements on the M. Havelaarweg and the Baarsweg,
\(^{4}\): first measurements on the Pascalweg,
\(^{5}\): no measurements on the Weiver.

In figure 5.2 the measured \(d_0\)-values on the Weiver in Zaanstad are presented. This section showed the largest deflections of all the test sections. Similar to figure 5.2, figure 5.3 shows the measured \(d_0\)-values on the Pascalweg in Rotterdam, which showed the smallest deflections of the seven test pavements. In appendix 5.1 similar figures are presented for the other test sections. In this appendix also the measured IDK.-values for all the test sections are presented.

As is shown by both the figures 5.2 and 5.3 the measured \(d_0\)-values show a decrease in time. Similar behaviour is found for the other five test sections, see appendix 5.1. This decrease in deflection was earlier found by the C.R.O.W working group D3 and formed the basis for the development of the "progressive stiffening" theory (14).

As is shown by figure 5.3 the largest deflections on the Pascalweg are measured in the right wheeltrack. To a smaller extent the same holds for the
Weiver, see figure 5.2. By considering appendix 5.1 it is found that this is a general trend found for six of the seven test sections.

**fig 5.2** Measured average $d_0$-values and coefficient of variation of $d_0$-values for the Weiver in Zaanstad.

**fig 5.3** Measured average $d_0$-values and coefficient of variation of $d_0$-values for the Pascalweg in Rotterdam.

For the IDK$_{0.3}$-values a similar, but weaker, trend is found. Again the IDK$_{0.3}$-values measured in the right wheeltrack are often larger than the one measured between the wheeltracks. The Weiver really is the only section in

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**Stiffness of the joints**

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which this tendency was not observed, see figure 5.4 and appendix 5.1.

**fig 5.4** Measured average IDK$_{0.3}$-values and coefficient of variation of IDK$_{0.3}$-values for the Weer in Zaanstad.

**fig 5.5** Measured average IDK$_{0.3}$-values and coefficient of variation of IDK$_{0.3}$-values for the Pascalweg in Rotterdam.

In figure 5.5 the measured IDK$_{0.3}$-values for the Pascalweg in Rotterdam are shown. The measurements performed on this section clearly show that the IDK$_{0.3}$ between the wheeltracks is smaller than the one measured in the right wheeltrack.

Stiffness of the joints
5.4 JOINT SPRING STIFFNESSES

5.4.1 The determination of the joint spring stiffnesses

As explained in the previous chapter the structural model of concrete block pavements requires the normal and shear stiffness of the springs representing the joints. The results of the previously discussed FWD-measurements are used to determine these stiffnesses.

The procedure followed to determine these spring stiffnesses started with the determination of the average deflection bowls measured on the seven pavements. Again a distinction was made between the deflection bowls measured in the right wheeltrack and the deflection bowls measured between the wheeltracks, so for each pavement two deflection bowls were obtained. Each of these deflection bowls was thus based upon $14 \times 11 = 154$ measured deflection bowls (for the Weiver only 143 deflection bowls are considered, see table 5.1).

These average measured deflection bowls were also calculated using the structural model discussed in the previous chapter. In these calculations all unbound granular materials used in the substructure of the various pavements were modelled by a constant Young's modulus, as was the subgrade. The Poisson's ratio for all these materials was fixed at 0.35 which is a commonly used value. The springs in the joints were only capable in transferring joint forces in the case of joint compression.

In the calculation procedure trial and error played a large role. By varying the joint spring stiffnesses and the Young's moduli of the various layers in the substructure a good agreement between the measured FWD-bowls and the calculated bowls was achieved. The procedure was ended when the best possible agreement between the measured FWD-bowl and the calculated deflections was found.

The procedure resulted in a set of 14 joint normal spring stiffnesses and 14 joint shear spring stiffnesses. Based on these two sets of spring stiffnesses the average normal joint spring stiffness was determined and proved to be 5,500 N/mm per mm modelled joint length. Similarly the average joint shear spring stiffness was determined, this stiffness equals 550 N/mm mm'. Both spring stiffnesses are valid for concrete block pavements in which the blocks have a thickness of 80 mm.

To get some idea of the properties of the material that is present in the joints of a concrete block layer a simple analysis on the basis of the elastic theory is made. Within this analysis it is assumed that the material in the
joints is subjected to only two types of strain. The first strain is a result of joint compression. The second strain is a shear strain that is a result of a difference in vertical displacement of the blocks next to the considered joint.

For the case of a pure joint compression of 1 mm, the following joint normal stress will develop.

\[ \sigma_n = \frac{(1-v)E}{(1+v)(1-2v)} \frac{1}{w} \] (5.2)

As a result of a pure 1 mm shear deformation over a joint (1 mm difference in vertical deflection of two adjacent blocks) the following shear stress will develop in the joint.

\[ \sigma_s = \frac{E}{2(1+v)} \frac{1}{w} \] (5.3)

where:
\( \sigma_n \): joint normal stress \([\text{N/mm}^2]\)
\( \sigma_s \): joint shear stress \([\text{N/mm}^2]\)
w: joint width \([\text{mm}]\)
E: Young’s modulus of the joint material \([\text{N/mm}^2]\)
v: Poisson’s ratio of the joint material \([-\text{]}\)

The stresses that develop as a result of a 1 mm joint deformation (compression or shear) as presented in the equations 5.2 and 5.3 act over the total joint. By multiplying the stresses with the thickness of the blocks the joint force per mm joint length per mm joint deformation is obtained. Since there are two sets of springs in each joint, this force \([\text{N/mm mm}]\) has to be divided by two to get the theoretically determined joint spring stiffness.

Assuming that the two described types of strain are the only strains that occur in the joint the theory of elasticity now prescribes that the Poisson’s ratio of the joint material must be 0.444 in order to make the compressive joint stiffness ten times larger than the shear joint stiffness.

The Young’s modulus combined with the joint width now determines the absolute values of the joint stiffnesses. The calculated joint spring stiffnesses are valid when the material in the joints has a Young’s modulus of 100 MPa in combination with 2.52 mm wide joints. Both these values seem very realistic (joint material is sand) which thus make the calculated spring stiffnesses more trustworthy.
5.4.2 Calculated deflection bowls

To give an idea of the accuracy that is achieved by the described back-calculation procedure the deflection bowls of the Weiver and the Pascalweg are presented here. In the calculations the mentioned average joint spring stiffnesses as discussed in the previous section are used. It is reminded here that the two pavements discussed here represent the pavement with the largest and the pavement with the smallest deflections.

Since the concrete blocks in the Weiver had a thickness of 70 mm the joint spring stiffnesses used for the back-calculated deflection bowls of this pavement structure equal 4812.5 N/mm mm$^2$ for the normal springs and 481.3 N/mm mm$^2$ for the shear springs. Both mentioned values thus equal $7/8$ times the values determined for 80 mm thick concrete blocks.

The calculated FWD-bowls shown in the figures 5.6 and 5.7 are all based on varying the Young’s moduli of four layers, the substructure of the two pavements was divided into three layers, whereas the fourth layer represented the subgrade. All the layers had a Poisson’s ratio of 0.35.

For the calculation of the FWD-bowl of the Weiver the substructure, which is the sand layer, is divided into three sub-layers. From top to bottom the thickness of these layers is 150 mm, 150 mm and 500 mm respectively. The stiffness of these layers is 40 MPa, 120 MPa and 230 MPa respectively for both the FWD-bowl between the wheeltracks and the FWD-bowl in the right wheeltrack. The difference between the two mentioned FWD-bowls was explained by the subgrade modulus which is 39.5 MPa for the FWD-bowl measured in the right wheel track and 42 MPa for the bowl measured between the wheeltracks.

For the calculation of the FWD-bowls measured on the Pascalweg the substructure was divided in three layers with the following thicknesses: 50 mm, 300 mm and 600 mm respectively. The upper layer, representing the bedding sand layer, has a 75 MPa modulus for the FWD-bowl measured in the right wheeltrack. For the 300 mm base, the 600 mm sub-base and the subgrade the stiffnesses become 1200 MPa, 475 MPa and 120 MPa respectively.

For the presented calculated FWD-bowl measured between the wheeltracks of the Pascalweg these stiffnesses become 125 MPa, 1500 MPa, 575 MPa and 142.5 MPa respectively.
fig 5.6  Calculated and measured average deflection bowls for the Weiver in Zaanstad.

fig 5.7  Calculated and measured average deflection bowls for the Pascalweg in Rotterdam.

To give an impression of the accuracy that is achieved the tables 5.2 and 5.3 are given. Table 5.2 gives the results of the calculation of the average
deflection bowls measured on the Weiver and table 5.3 gives the similar results for the Pascalweg.

<table>
<thead>
<tr>
<th>in right wheeltrack:</th>
<th>( d_0 )</th>
<th>( d_{0.3} )</th>
<th>( d_{0.6} )</th>
<th>( d_{0.9} )</th>
<th>( d_{1.2} )</th>
<th>( d_{1.8} )</th>
<th>IDK_{0.3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>measured [( \mu \text{m} )]</td>
<td>1127.6</td>
<td>717.4</td>
<td>405.1</td>
<td>296.0</td>
<td>234.1</td>
<td>198.8</td>
<td>410.2</td>
</tr>
<tr>
<td>calculated [( \mu \text{m} )]</td>
<td>1123.5</td>
<td>714.3</td>
<td>376.0</td>
<td>299.9</td>
<td>252.9</td>
<td>191.7</td>
<td>409.2</td>
</tr>
<tr>
<td>difference in [% [-]</td>
<td>-0.4</td>
<td>-0.4</td>
<td>-7.2</td>
<td>1.3</td>
<td>8.0</td>
<td>-3.6</td>
<td>-0.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>between wheeltracks:</th>
<th>( d_0 )</th>
<th>( d_{0.3} )</th>
<th>( d_{0.6} )</th>
<th>( d_{0.9} )</th>
<th>( d_{1.2} )</th>
<th>( d_{1.8} )</th>
<th>IDK_{0.3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>measured [( \mu \text{m} )]</td>
<td>1100.2</td>
<td>660.7</td>
<td>376.3</td>
<td>284.8</td>
<td>226.7</td>
<td>181.6</td>
<td>439.5</td>
</tr>
<tr>
<td>calculated [( \mu \text{m} )]</td>
<td>1106.2</td>
<td>696.7</td>
<td>359.4</td>
<td>284.8</td>
<td>239.2</td>
<td>180.5</td>
<td>409.5</td>
</tr>
<tr>
<td>difference in [% [-]</td>
<td>0.5</td>
<td>5.4</td>
<td>-4.5</td>
<td>0.0</td>
<td>5.5</td>
<td>-0.6</td>
<td>-6.8</td>
</tr>
</tbody>
</table>

Table 5.2 Results of the calculation of the average deflection bowls measured on the Weiver in Zaanstad.

<table>
<thead>
<tr>
<th>in right wheeltrack:</th>
<th>( d_0 )</th>
<th>( d_{0.3} )</th>
<th>( d_{0.6} )</th>
<th>( d_{0.9} )</th>
<th>( d_{1.2} )</th>
<th>( d_{1.8} )</th>
<th>IDK_{0.3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>measured [( \mu \text{m} )]</td>
<td>356.4</td>
<td>192.6</td>
<td>128.6</td>
<td>103.6</td>
<td>87.9</td>
<td>70.0</td>
<td>163.8</td>
</tr>
<tr>
<td>calculated [( \mu \text{m} )]</td>
<td>357.4</td>
<td>202.2</td>
<td>129.0</td>
<td>104.8</td>
<td>88.5</td>
<td>65.6</td>
<td>155.2</td>
</tr>
<tr>
<td>difference in [% [-]</td>
<td>0.3</td>
<td>5.0</td>
<td>0.3</td>
<td>1.2</td>
<td>0.7</td>
<td>-6.3</td>
<td>-5.3</td>
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</table>

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<thead>
<tr>
<th>between wheeltracks:</th>
<th>( d_0 )</th>
<th>( d_{0.3} )</th>
<th>( d_{0.6} )</th>
<th>( d_{0.9} )</th>
<th>( d_{1.2} )</th>
<th>( d_{1.8} )</th>
<th>IDK_{0.3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>measured [( \mu \text{m} )]</td>
<td>277.8</td>
<td>150.9</td>
<td>103.4</td>
<td>87.7</td>
<td>76.2</td>
<td>61.8</td>
<td>126.9</td>
</tr>
<tr>
<td>calculated [( \mu \text{m} )]</td>
<td>283.7</td>
<td>162.0</td>
<td>109.2</td>
<td>89.6</td>
<td>75.7</td>
<td>55.8</td>
<td>121.7</td>
</tr>
<tr>
<td>difference in [% [-]</td>
<td>2.1</td>
<td>7.3</td>
<td>5.6</td>
<td>2.2</td>
<td>-0.7</td>
<td>-9.7</td>
<td>-4.1</td>
</tr>
</tbody>
</table>

Table 5.3 Results of the calculation of the average deflection bowls measured on the Pascalweg in Rotterdam.

As shown by the figures 5.6 and 5.7 and the tables 5.2 and 5.3 the calculated deflection bowls quite closely agree with the average measured FWD-bowls. It is stated here that even better fits can be achieved by also varying the joint spring stiffnesses. Especially the agreement between the measured and calculated \( d_{0.3} \) and as a result the IDK_{0.3} will improve by also

Stiffness of the joints

87
varying the joint spring stiffnesses. The calculations discussed here are however intended to show the effects of the joint spring stiffnesses that are used in the further analyses of concrete block pavements.
Appendix 5.1
Falling Weight Deflection measurement results

**fig a5.1.1** Measured average \(d_0\)-values and coefficient of variation of \(d_0\)-values for the M. Havelaarweg in Rotterdam.

**fig a5.1.2** Measured average \(IDK_{0.3}\)-values and coefficient of variation of \(IDK_{0.3}\)-values for the M. Havelaarweg in Rotterdam.

Appendix 5.1: Falling Weight Deflection measurement results 89
fig a5.1.3  Measured average $d_0$-values and coefficient of variation of $d_0$-values for the Baarsweg in Rotterdam.

fig a5.1.4  Measured average $IDK_{0.3}$-values and coefficient of variation of $IDK_{0.3}$-values for the Baarsweg in Rotterdam.
fig a5.1.5 Measured average $d_0$-values and coefficient of variation of $d_0$-values for the Pascalweg in Rotterdam.

fig a5.1.6 Measured average $IDK_{0.3}$-values and coefficient of variation of $IDK_{0.3}$-values for the Pascalweg in Rotterdam.
fig a.5.1.7 Measured average $d_0$-values and coefficient of variation of $d_0$-values for the Zaanweg in Zaanstad.

fig a.5.1.8 Measured average $IDK_{0.3}$-values and coefficient of variation of $IDK_{0.3}$-values for the Zaanweg in Zaanstad.
fig a5.1.9 Measured average $d_0$-values and coefficient of variation of $d_0$-values for the C. Bruynweg in Zaanstad.

fig a5.1.10 Measured average $IDK_{0.3}$-values and coefficient of variation of $IDK_{0.3}$-values for the C. Bruynweg in Zaanstad.
fig a5.1.11 Measured average $d_c$-values and coefficient of variation of $d_c$-values for the Weiver in Zaanstad.

fig a5.1.12 Measured average $IDK_{0.3}$-values and coefficient of variation of $IDK_{0.3}$-values for the Weiver in Zaanstad.
**Fig a5.1.13** Measured average $d_0$-values and coefficient of variation of $d_0$-values for the Allanstraat in Zaanstad.

**Fig a5.1.14** Measured average $IDK_{0.3}$-values and coefficient of variation of $IDK_{0.3}$-values for the Allanstraat in Zaanstad.
6 Material Testing

6.1 INTRODUCTION

The structural model described in chapter four enables the resilient analysis of a concrete block pavement structure. For such an analysis the resilient properties of the materials applied in the concrete block pavement have to be known. In the previous chapter the properties of the jointing material, expressed in joint spring stiffnesses, were determined and discussed. In this chapter the properties of eight sands and four base course materials are discussed. Furthermore the laboratory tests performed to obtain these properties are described.

All materials that are involved in this research were taken from the substructures from seven in-service concrete block pavements described in the previous chapter: Zaanweg, Cor Bruynweg, Allanstraat and Weiver in the city of Zaanstad and Pascalweg, Baarsweg and Max Havelaarweg in the city of Rotterdam.

From all these pavement structures a sample of the sub-base sand was taken and from the Max Havelaarweg also a sample of the crusher sand used in the bedding layer. Four of these pavement structures have a base layer, i.e. C. Bruynweg, Allanstraat, Pascalweg and the M. Havelaarweg. Also samples were taken from these base layers.

6.2 SIEVE ANALYSIS

6.2.1 Test description

Sieve analyses were done to get insight into the particle size distribution of the granular materials. The sieving was conducted according to the Dutch specifications (31). For the natural sands, i.e. the sub-base sands, the following sieves ($\phi=200$ mm) have been used: 0.063, 0.125, 0.180, 0.250, 0.355, 0.500, 1 and 2 mm. For the coarser crusher sand the 2.8, 4, 5.6 and 8
mm sieves were added. All sands have been dry sieved and wet sieved.

For the base course materials the sieves up to 2 mm equal the sieves used for the sub-base sands. The following sieves (ϕ=350 mm) have been added: 2.8, 4, 5.6, 8, 11.2, 16, 22.4, 31.5 and 45 mm.

Also on the base materials two sieve analyses were performed. The first was according to the Dutch specifications (31), while the second sieve analysis was performed on a much larger quantity of material as needed to perform a crushing resistance test. All base course materials have been dry sieved.

6.2.2 Test results

In the tables 6.1 and 6.2 the results of the sieve analysis for the sands are presented.

<table>
<thead>
<tr>
<th>Sand</th>
<th>Zaanweg</th>
<th>Weiver</th>
<th>Allanstraat</th>
<th>C. Bruynweg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve [mm]</td>
<td>dry</td>
<td>wet</td>
<td>dry</td>
<td>wet</td>
</tr>
<tr>
<td>2</td>
<td>99.6</td>
<td>98.3</td>
<td>99.5</td>
<td>98.2</td>
</tr>
<tr>
<td>1</td>
<td>98.5</td>
<td>97.2</td>
<td>98.6</td>
<td>97.4</td>
</tr>
<tr>
<td>0.5</td>
<td>95.3</td>
<td>93.8</td>
<td>97.0</td>
<td>96.2</td>
</tr>
<tr>
<td>0.355</td>
<td>89.8</td>
<td>87.8</td>
<td>93.5</td>
<td>94.2</td>
</tr>
<tr>
<td>0.25</td>
<td>53.9</td>
<td>54.1</td>
<td>57.1</td>
<td>55.8</td>
</tr>
<tr>
<td>0.180</td>
<td>15.6</td>
<td>21.7</td>
<td>16.0</td>
<td>18.5</td>
</tr>
<tr>
<td>0.125</td>
<td>5.6</td>
<td>9.3</td>
<td>3.3</td>
<td>5.3</td>
</tr>
<tr>
<td>0.063</td>
<td>1.3</td>
<td>3.7</td>
<td>0.9</td>
<td>2.8</td>
</tr>
</tbody>
</table>

*Table 6.1* Results of the dry and wet sieve analysis for the Zaanstad sands.
<table>
<thead>
<tr>
<th>Sand [mm]</th>
<th>Crusher</th>
<th>Baarsweg</th>
<th>Pascalweg</th>
<th>M. Havelaarweg</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>dry</td>
<td>wet</td>
<td>dry</td>
<td>wet</td>
</tr>
<tr>
<td>5.6</td>
<td>91.5</td>
<td>94.9</td>
<td>97.8</td>
<td>97.7</td>
</tr>
<tr>
<td>4</td>
<td>81.6</td>
<td>85.3</td>
<td>97.3</td>
<td>99.0</td>
</tr>
<tr>
<td>2.8</td>
<td>68.2</td>
<td>74.2</td>
<td>95.8</td>
<td>96.9</td>
</tr>
<tr>
<td>2</td>
<td>54.8</td>
<td>59.4</td>
<td>93.9</td>
<td>94.4</td>
</tr>
<tr>
<td>1</td>
<td>35.3</td>
<td>37.3</td>
<td>88.3</td>
<td>87.2</td>
</tr>
<tr>
<td>0.5</td>
<td>22.5</td>
<td>23.2</td>
<td>76.9</td>
<td>76.0</td>
</tr>
<tr>
<td>0.355</td>
<td>17.8</td>
<td>18.9</td>
<td>24.3</td>
<td>30.8</td>
</tr>
<tr>
<td>0.25</td>
<td>12.1</td>
<td>13.4</td>
<td>0.9</td>
<td>4.9</td>
</tr>
<tr>
<td>0.180</td>
<td>7.9</td>
<td>9.5</td>
<td>4.9</td>
<td>9.0</td>
</tr>
<tr>
<td>0.125</td>
<td>5.1</td>
<td>7.0</td>
<td>0.4</td>
<td>9.0</td>
</tr>
</tbody>
</table>

Table 6.2  Results of the dry and wet sieve analysis for the Rotterdam sands.

In Table 6.3 the results of the dry sieve analysis for the base course materials are presented. Due to a lack of the Pascalweg base course material only the sieve test according the Dutch specifications (31) was performed for this base material.

Also graphical representations of the grain size distributions are made. In these plots the Fuller curve is also given. The Fuller curve is based on equation 6.1 and results in the theoretical best packing for a material consisting of spheres with diameters which equal the sieve sizes.

\[
\text{Percentage passing} = 100 \times \sqrt[3]{\frac{D}{D_{\text{max}}}}
\]  

(6.1)

where:

\(D\):

Diameter of the spheres [mm]

\(D_{\text{max}}\):

Maximum diameter of the spheres in the material [mm]
<table>
<thead>
<tr>
<th>Sieve [mm]</th>
<th>Percentage passing [% m/m]</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>45</td>
<td>98.4</td>
<td>98.8</td>
<td>97.1</td>
<td>96.7</td>
<td>97.7</td>
<td>97.7</td>
</tr>
<tr>
<td>31.5</td>
<td>94.2</td>
<td>93.2</td>
<td>82.7</td>
<td>84.2</td>
<td>93.9</td>
<td>91.9</td>
</tr>
<tr>
<td>22.4</td>
<td>87.2</td>
<td>85.9</td>
<td>67.2</td>
<td>69.8</td>
<td>90.6</td>
<td>85.9</td>
</tr>
<tr>
<td>16</td>
<td>79.3</td>
<td>78.1</td>
<td>55.6</td>
<td>58.4</td>
<td>85.6</td>
<td>80.3</td>
</tr>
<tr>
<td>11.2</td>
<td>70.1</td>
<td>70.2</td>
<td>46.6</td>
<td>49.3</td>
<td>78.8</td>
<td>72.2</td>
</tr>
<tr>
<td>8</td>
<td>62.3</td>
<td>64.7</td>
<td>40.0</td>
<td>43.7</td>
<td>70.7</td>
<td>64.1</td>
</tr>
<tr>
<td>5.6</td>
<td>57.4</td>
<td></td>
<td>35.7</td>
<td></td>
<td>61.7</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>54.2</td>
<td>55.5</td>
<td>32.2</td>
<td>36.5</td>
<td>54.9</td>
<td>47.7</td>
</tr>
<tr>
<td>2.8</td>
<td>51.4</td>
<td></td>
<td>30.3</td>
<td></td>
<td>49.9</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>48.8</td>
<td>48.9</td>
<td>28.0</td>
<td>31.2</td>
<td>45.2</td>
<td>36.7</td>
</tr>
<tr>
<td>1</td>
<td>42.9</td>
<td>44.4</td>
<td>23.8</td>
<td>27.2</td>
<td>37.1</td>
<td>28.8</td>
</tr>
<tr>
<td>0.5</td>
<td>35.9</td>
<td>38.4</td>
<td>19.7</td>
<td>22.6</td>
<td>26.2</td>
<td>20.4</td>
</tr>
<tr>
<td>0.25</td>
<td>22.1</td>
<td>24.8</td>
<td>10.8</td>
<td>13.7</td>
<td>11.3</td>
<td>9.7</td>
</tr>
<tr>
<td>0.125</td>
<td>4.2</td>
<td>5.1</td>
<td>2.8</td>
<td>3.5</td>
<td>5.1</td>
<td>3.8</td>
</tr>
<tr>
<td>0.063</td>
<td>0.8</td>
<td>1.1</td>
<td>1.6</td>
<td>1.7</td>
<td>1.1</td>
<td>1.4</td>
</tr>
</tbody>
</table>

**Table 6.3** Results of the dry sieve analysis for base course materials,
1: According to Dutch specifications (31), 2: Larger quantity of material.

Figures 6.1 and 6.2 give the grain size distributions found for the sands together with the Fuller curves. The figures show that the seven natural sands have a very uniform grain size distribution, the crusher sand however shows a continuous distribution which almost equals the Fuller curve.

According to the Dutch specifications for sub-base sands, the sand applied at a depth of less than 1 m below the road surface should be a mineral material, of which the percentage passing the 63 μm sieve should be less than 15% m/m. If this percentage is between 10% m/m and 15% m/m then an additional demand is formulated. In this case the percentage passing the 20 μm sieve should be 3% m/m at most. As is shown by the figures 6.1 and 6.2 and the tables 6.1 and 6.2 all seven sands completely comply to these specifications.

The grain size distributions found for the base course materials are presented in figure 6.3. As is shown in this figure these materials show a continuous distribution. The Fuller curve is also plotted in the figure.
In figure 6.3 the Dutch specifications for 0/40 base course materials are plotted as well. Both the Zaanstad base course materials do not completely comply to these specifications. The Allanstraat base course material is exceeding the upper limit at the grain sizes between 0.25 mm and 2.8 mm. This means that this material shows a shortage of grains with these sizes. The C. Bruynweg base material is exceeding the lower limit between the grain sizes of 5.6 mm to 22.4 mm and thus has too many grains with a size between these limits.
6.3 GRAIN SHAPE

6.3.1 Test description

The grains of a granular material can have various shapes, see figure 6.4. Volders and Verhoeven developed an outflow test which expresses the shape of the grains into a figure, the Volders and Verhoeven Sharpness, "VVS" (32). The lower this VVS, the more rounded the grains of the tested material are. The equipment needed to perform the test is very simple, the test procedure itself is somewhat more complex, see appendix 6.1.

VVS= 0 %  VVS= 100 %

fig 6.4 Grain shapes.
The VVS relates the shape of the grains of the tested material to the shape of two reference materials. A VVS of 0% is obtained when the grain shape of the tested material equals the grain shape of reference glass pearls. A VVS of 100% is found when the tested material has grains that resemble the grains of a reference crusher sand.

### 6.3.2 Test results

As discussed in appendix 6.1, the determination of the sharpness of a material requires the specific gravity of the grain material. In table 6.4 the specific gravity of the grains of the various materials is given. For the base course materials the specific gravity is determined on the fraction < 5.6 mm due to the physical limitations of the pycnometer, see appendix 6.1.

<table>
<thead>
<tr>
<th>Zaanstad sand</th>
<th>Specific gravity [kg/m³]</th>
<th>Rotterdam sand</th>
<th>Specific gravity [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zaanweg sand</td>
<td>2635</td>
<td>Baarsweg sand</td>
<td>2657</td>
</tr>
<tr>
<td>Weiver sand</td>
<td>2649</td>
<td>M. Havelaarweg sand</td>
<td>2645</td>
</tr>
<tr>
<td>Allansstraat sand</td>
<td>2633</td>
<td>Pascalweg sand</td>
<td>2641</td>
</tr>
<tr>
<td>C. Bruynweg sand</td>
<td>2637</td>
<td>Crusher sand</td>
<td>2673</td>
</tr>
<tr>
<td>Zaanstad base</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Allansstraat base</td>
<td>2612</td>
<td>M. Havelaarweg base</td>
<td>2655</td>
</tr>
<tr>
<td>C. Bruynweg base</td>
<td>2613</td>
<td>Pascalweg base</td>
<td>2644</td>
</tr>
</tbody>
</table>

*Table 6.4 Specific gravity of the sand grains and the base material grains (< 5.6 mm).*

The sharpness of the sands was determined on 600 grams of grinded material. Use was made of a rubber stamper so that grain conglomerations were destroyed while the individual grains remained intact. The obtained results are given in table 6.5.

<table>
<thead>
<tr>
<th>Zaanstad sand</th>
<th>VVS [-]</th>
<th>Rotterdam sand</th>
<th>VVS [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zaanweg sand</td>
<td>72.13%</td>
<td>Baarsweg sand</td>
<td>74.29%</td>
</tr>
<tr>
<td>Weiver sand</td>
<td>72.85%</td>
<td>M. Havelaarweg sand</td>
<td>72.40%</td>
</tr>
<tr>
<td>Allansstraat sand</td>
<td>66.52%</td>
<td>Pascalweg sand</td>
<td>66.59%</td>
</tr>
<tr>
<td>C. Bruynweg sand</td>
<td>94.11%</td>
<td>Crusher sand</td>
<td>115.74%</td>
</tr>
</tbody>
</table>

*Table 6.5 Volders and Verhoeven Sharpness of the sands.*
As shown by table 6.5 the crusher sand involved in this research is sharper than the crusher sand used as a reference material by Volders and Verhoeven, resulting in a sharpness larger than 100%.

The sharpness of the base course materials was determined on the fraction < 2 mm. Since these base materials might show cemented grains it was decided not to grind the material using a rubber stamper. Grinding would destroy grain conglomerations which should remain intact since they are part of the material as such. In order to separate grains in a non original grain conglomeration the material was washed through the sieves. After this wet sieving the material was dried and tested. In an additional VVS-test dry sieved material was tested.

The results of the VVS-tests performed on the base course materials are presented in table 6.6. As shown by this table the VVS determined on washed material is larger than the VVS determined on the dry sieved material for two out of four materials. For the other two materials the situation is the other way around.

The base course materials all show relatively high VVS-values. Again VVS-values larger than 100% are obtained.

<table>
<thead>
<tr>
<th>Base course material</th>
<th>VVS_{\text{dry}} [-]</th>
<th>VVS_{\text{washed}} [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>92.86 %</td>
<td>82.78 %</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>117.37 %</td>
<td>106.84 %</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>105.01 %</td>
<td>128.79 %</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>94.98 %</td>
<td>125.55 %</td>
</tr>
</tbody>
</table>

*table 6.6  Sharpness of the base course materials (grains < 2 mm).*

### 6.4 MOISTURE-DENSITY AND MOISTURE-CBR RELATION

#### 6.4.1 Test description

Amongst other factors, the behaviour of granular materials is influenced by the moisture content of the material. To get insight into the effects of the moisture content on the compactability, the material can be compacted by means of a standard procedure in which the amount of energy used for compaction is fixed. This well known Proctor test, see appendix 6.2, is performed at various moisture contents and thus gives insight into the
moisture content at which the material is best compactable and reaches optimum compaction.

Apart from the compactability, the bearing capacity of a granular material also depends on the moisture content (and the density of the material). The relation between moisture content and the bearing capacity of the sands was tested by performing CBR-tests (Californian Bearing Ratio). The well known CBR-test is described in appendix 6.2. The test expresses the bearing capacity of a material as a percentage of the bearing capacity of a reference crushed rock.

6.4.2 Test results

By plotting the test results against the moisture content insight is obtained into the moisture-density and moisture-CBR relation of a sand. In figure 6.5 such a plot is given, similar figures for all the tested sands can be found in appendix 6.2. During the tests the moisture content was increased until the sample loosed water during compaction, so the last data points represent a completely saturated sample.

![Figure 6.5: Moisture-density and moisture-CBR relationships for the Zaanweg sub-base sand.](image)

Table 6.7 gives the results of the tests performed on the sands. In this table the maximum dry density, $\gamma_{\text{dry, max}}$, is given. The moisture content at which this density was obtained is also given, $W_{\text{opt}}$. This optimum moisture content is thus related to the maximum dry density of the material and not to the maximum CBR-value.

If a material showed two moisture contents at which a maximum dry
density, $\gamma_{\text{dry,max}}$, was found then $W_{\text{opt}}$ refers to the moisture content at the first maximum, i.e. the lowest moisture content. Another option is that the material shows an increasing dry density with an increasing moisture content. In that case the maximum dry density was found at the last test that was performed, i.e. a test in which the sample loosed water. The values of both $\gamma_{\text{dry,max}}$ and $W_{\text{opt}}$ in these cases refer to the highest moisture content at which the material did not loose water.

<table>
<thead>
<tr>
<th>Sand</th>
<th>$\gamma_{\text{dry,max}}$ [kg/m$^3$]</th>
<th>$W_{\text{opt}}$ [-]</th>
<th>$\gamma_{\text{wet}}$ [kg/m$^3$]</th>
<th>CBR [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crusher</td>
<td>1755</td>
<td>10.5 %</td>
<td>1937</td>
<td>15.7 %</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>1668</td>
<td>13.5 %</td>
<td>1892</td>
<td>11.0 %</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>1592</td>
<td>14.5 %</td>
<td>1820</td>
<td>11.0 %</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>1648</td>
<td>14.5 %</td>
<td>1882</td>
<td>12.1 %</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>1701</td>
<td>12.5 %</td>
<td>1915</td>
<td>21.7 %</td>
</tr>
<tr>
<td>C. Bruynstraat</td>
<td>1723</td>
<td>12.5 %</td>
<td>1942</td>
<td>22.0 %</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>1689</td>
<td>14.0 %</td>
<td>1926</td>
<td>21.1 %</td>
</tr>
<tr>
<td>Weiver</td>
<td>1706</td>
<td>13.0 %</td>
<td>1928</td>
<td>20.7 %</td>
</tr>
</tbody>
</table>

*Table 6.7* Some characteristics retrieved from the moisture-density and moisture-CBR relationships for the sands.

The CBR-value given in table 6.7 refers to the maximum CBR measured on non-saturated samples. This value is not necessarily equal to the CBR-value measured at $W_{\text{opt}}$ which is related to $\gamma_{\text{dry,max}}$.

No CBR-tests are performed on the base course materials. An example of the moisture-density relation measured for the base materials is given in figure 6.6, for the other base course material test results see appendix 6.2. In table 6.8 the results for the base course materials are summarized.

<table>
<thead>
<tr>
<th>Base course material</th>
<th>$\gamma_{\text{dry,max}}$ [kg/m$^3$]</th>
<th>$W_{\text{opt}}$ [-]</th>
<th>$\gamma_{\text{wet}}$ [kg/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M. Havelaarweg</td>
<td>1632</td>
<td>15.2 %</td>
<td>1880</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>1929</td>
<td>9.1 %</td>
<td>2104</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>1885</td>
<td>10.5 %</td>
<td>2083</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>1679</td>
<td>14.9 %</td>
<td>1929</td>
</tr>
</tbody>
</table>

*Table 6.8* Wet and dry densities combined with the optimum moisture content for the base course materials.
As a result of the large maximum grain size of the base materials the Proctor test could not be performed on the total material as such. Depending on the grain size distribution of the base material the fraction > 31.5 mm or the fraction > 45 mm was excluded from the actual Proctor tests. By performing some additional tests on the excluded grains it was however possible to correct the results of the Proctor tests and take into account the effects of the larger excluded grains, see appendix 6.2. In figure 6.6 the results of the Proctor test performed on the Pascalweg base material are presented whereas table 6.8 gives the corrected test results.

![Graph showing moisture-density relationship for Pascalweg base material.](image)

**fig 6.6** Moisture-density relationship for the Pascalweg base material.

### 6.5 STATIC TRIAXIAL TEST

#### 6.5.1 Test description

In the static triaxial test a granular material is subjected to a controlled confining stress, $\sigma_{\text{conf}}$. This all around confining stress does not introduce any shear stress in the material. Apart from $\sigma_{\text{conf}}$ the material is also subjected to a slowly increasing additional stress applied by an actuator in a displacement controlled mode. The sum of the confining stress, the vertical stress as a result of dead weight and the additional stress applied by the actuator, $\sigma_{\text{act}}$, is the first principal stress, $\sigma_1$.

Due to the slowly increasing actuator stress $\sigma_{\text{act}}$, also $\sigma_1$ will slowly increase. As the sample deformation becomes too large, $\sigma_1$ decreases after reaching its maximum value. This maximum value is referred to as the first...
principal stress at failure, $\sigma_{1,f}$, which depends on $\sigma_{\text{conf}}$. A more detailed explanation of the static triaxial test is given in appendix 6.3.

### 6.5.2 Test results

Per material at least three static triaxial tests to failure were performed. The $\sigma_{\text{conf}}$-level was different in these tests. For the sands three newly build samples were used in these tests. Due to the time required for building a large base course sample, the tests performed on base course materials mostly concerned one virgin sample, see appendix 6.3. Two additional tests were performed on samples which had been tested for other purposes (i.e. resilient triaxial test and low stress-level permanent strain triaxial test).

The results of the three tests show that $\sigma_{1,f}$ increases with $\sigma_{\text{conf}}$. This increase of $\sigma_{1,f}$ is explained by the failure criterion of Mohr-Coulomb from which the following expression for $\sigma_{1,f}$ can be retrieved.

$$\sigma_{1,f} = \frac{(1 + \sin \phi) \sigma_{\text{conf}} + 2c \cos \phi}{1 - \sin \phi}$$  \hspace{1cm} (6.2)

where:
- $c$: cohesion of the material [kPa]
- $\phi$: angle of internal friction of the material [$^\circ$]
- $\sigma_{\text{conf}}$: confining pressure [kPa]

For the purpose of linear regression this equation is rewritten into:

$$\sigma_{1,f} - A \sigma_{\text{conf}} = B$$

in which:

$$A = \frac{1 + \sin \phi}{1 - \sin \phi} \quad \text{and} \quad B = \frac{2c \cos \phi}{1 - \sin \phi}$$  \hspace{1cm} (6.3)

The values of $A$ and $B$ can now be determined on the basis of the test results using the least squares method. Hereafter $A$ and $B$ are translated to $c$ and $\phi$. For the sands the fits between the data and the model (equation 6.3) are almost perfect ($r^2 > 0.99$ in all cases). An example of the results obtained for the sands is given in figure 6.7. Similar plots obtained for the other sands can be found in appendix 6.3. In table 6.9 the $c$- and $\phi$- values found for the sands are given.
**fig 6.7** Results of three static failure triaxial tests for the Zaanweg sand.

<table>
<thead>
<tr>
<th>Zaanstad sands</th>
<th>c [kPa]</th>
<th>φ [°]</th>
<th>Rotterdam sands</th>
<th>c [kPa]</th>
<th>φ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zaanweg</td>
<td>4.08</td>
<td>43.9</td>
<td>Baarsweg</td>
<td>7.19</td>
<td>42.8</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>5.60</td>
<td>48.2</td>
<td>M. Havelaarweg</td>
<td>7.48</td>
<td>42.9</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>6.34</td>
<td>41.8</td>
<td>Pascalweg</td>
<td>7.99</td>
<td>39.7</td>
</tr>
<tr>
<td>Weiver</td>
<td>6.76</td>
<td>43.0</td>
<td>Crusher</td>
<td>8.68</td>
<td>50.2</td>
</tr>
</tbody>
</table>

**table 6.9** Cohesion and angle of internal friction found for the sands.

In figure 6.8 an example is given of the results of the static triaxial tests performed on the base course materials, the plots for the other base materials are given in appendix 6.3. The c- and φ-values obtained for the base course materials are given in table 6.10.

<table>
<thead>
<tr>
<th>Zaanstad bases</th>
<th>c [kPa]</th>
<th>φ [°]</th>
<th>Rotterdam bases</th>
<th>c [kPa]</th>
<th>φ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>45.89</td>
<td>41.6</td>
<td>Pascalweg</td>
<td>68.67</td>
<td>51.0</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>58.84</td>
<td>49.6</td>
<td>M. Havelaarweg</td>
<td>73.69</td>
<td>43.7</td>
</tr>
</tbody>
</table>

**table 6.10** Cohesion and angle of internal friction found for the base course materials.
6.6 RESILIENT STRAIN CYCLIC TRIAXIAL TEST

6.6.1 Test description

For testing the resilient behaviour of granular materials under cyclic loading use was made of triaxial test set ups. Similar to the static test again the sample is first subjected to a confining stress, \( \sigma_{\text{conf}} \). After \( \sigma_{\text{conf}} \) is applied the actuator, in force controlled mode, is used to apply a minor static vertical stress, \( \sigma_{\text{stat}} \). Next to that the actuator applies a half sine shaped cyclic stress, \( \sigma_{\text{cyl}} \), at a frequency of 1 Hz.

As a result of \( \sigma_{\text{cyl}} \) axial and radial deformations will occur. The major part of these deformations is resilient and from these resilient deformations the stiffness modulus and the Poisson’s ratio are calculated.

Since granular materials show a stress dependent behaviour the cyclic triaxial test is performed at various stress combinations. In all cases the tests are started at a low \( \sigma_{\text{conf}} \)-level and a low \( \sigma_{\text{cyl}} \)-level. After the first measurements the \( \sigma_{\text{cyl}} \)-level is increased and another measurement is started. When a high enough \( \sigma_{\text{cyl}} \)-level is reached, \( \sigma_{\text{conf}} \) is increased. Then a new series of measurements using increasing \( \sigma_{\text{cyl}} \)-levels is started. This sequence of testing is repeated until all the desired data are obtained.
For a more detailed description of the resilient strain cyclic triaxial test one is referred to appendix 6.4.

### 6.6.2 Test results

As mentioned earlier the resilient axial strain, $\varepsilon_{r,ax}$, and the resilient radial strain, $\varepsilon_{r,rad}$, as a result of the cyclic stress, $\sigma_{cyc}$, are measured. From these strains the resilient modulus, $Mr$, and the Poisson’s ratio, $\nu$, are calculated using the following equations:

$$Mr = \frac{\sigma_{cyc}}{1000 \times \varepsilon_{r,ax}}$$  \hspace{1cm} (6.4)

$$\nu = -\frac{\varepsilon_{r,rad}}{\varepsilon_{r,ax}}$$  \hspace{1cm} (6.5)

where:
- $Mr$: resilient modulus [MPa]
- $\nu$: Poisson’s ratio [-]
- $\sigma_{cyc}$: maximum cyclic actuator stress [kPa]
- $\varepsilon_{r,ax}$, $\varepsilon_{r,rad}$: peak value of the axial and radial resilient strain respectively [-]

The most commonly used model to describe the stress dependency of $Mr$ is the $Mr-\Theta$ model introduced by Brown and Pell (33). The $Mr-\Theta$ model describes the measured $Mr$ data by a straight line when plotted against the sum of the principal stresses, $\Theta$, on a log-log scale.

The principal stresses in the triaxial test are the following:

$$\sigma_1 = \sigma_{conf} + \sigma_{cyc} + \sigma_{stat} + \sigma_{dw}$$  \hspace{1cm} (6.6)

$$\sigma_2 = \sigma_{conf}$$

$$\sigma_3 = \sigma_{conf}$$

The sum of these principal stresses is:

$$\Theta = \sigma_1 + \sigma_2 + \sigma_3$$  \hspace{1cm} (6.7)

where:
- $\sigma_{conf}$: confining stress [kPa]
- $\sigma_{cyc}$: maximum cyclic actuator stress [kPa]
- $\sigma_{stat}$: static actuator stress [kPa]
- $\sigma_{dw}$: static stresses as a result of dead weight [kPa]
The Mr-Ω model is given by equation 6.8.

\[ Mr = k1 \left( \frac{\Theta}{\Theta_0} \right)^{k2} \]  

(6.8)

where:

- \( k1 \): model parameter [MPa]
- \( k2 \): model parameter [-]
- \( \Theta \): sum of the principal stresses [kPa]
- \( \Theta_0 \): reference stress of 1 kPa [kPa]

The Mr-Ω model fitted the Mr data measured on the eight sands fairly good if one considers the data set in a general way. An example hereof is given in figure 6.9. For the other sands similar results were obtained, see appendix 6.4 and table 6.11. Although figure 6.9 and table 6.11 show that the Mr-Ω model describes the general trend in the data quite well, one arrives to a completely different conclusion if the various levels of \( \sigma_{\text{conf}} \) are considered. Per \( \sigma_{\text{conf}} \) level the increase of \( \Theta \) completely depends on an increase of \( \sigma_1 \). For a constant level of \( \sigma_{\text{conf}} \), the data in figure 6.9 clearly shows a decreasing Mr with an increasing \( \Theta \). An increase of \( \sigma_1 \) thus leads to a decrease of the Mr of sands. The same data however shows that an increase in \( \sigma_{\text{conf}} \) leads to an increase of Mr.

![Graph showing Mr vs Θ for different constants](image.png)

**fig 6.9** The Mr-Ω model combined with the data obtained for the Zaanweg sub-base sand.

It might be clear that the Mr-Ω model is not able to describe this behaviour since an increase of both \( \sigma_{\text{conf}} \) and \( \sigma_1 \) lead to an increase of \( \Theta \) and
thus to an increase of the modelled Mr. Therefore a new model was developed that is capable to describe the effects of both $\sigma_{\text{conf}}$ and $\sigma_1$ on the development of Mr; this model is given by equation 6.9.

<table>
<thead>
<tr>
<th>sand</th>
<th>$k1$ [MPa]</th>
<th>$k2$ [-]</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>18.6</td>
<td>0.48</td>
<td>0.81</td>
</tr>
<tr>
<td>Weiver</td>
<td>10.1</td>
<td>0.60</td>
<td>0.82</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>12.6</td>
<td>0.55</td>
<td>0.84</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>6.3</td>
<td>0.72</td>
<td>0.77</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>7.4</td>
<td>0.62</td>
<td>0.85</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>5.4</td>
<td>0.69</td>
<td>0.82</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>8.2</td>
<td>0.67</td>
<td>0.79</td>
</tr>
<tr>
<td>Crusher sand</td>
<td>5.7</td>
<td>0.55</td>
<td>0.88</td>
</tr>
</tbody>
</table>

*Table 6.11 Parameters found for the Mr-$\Theta$ model for sands.*

$$\text{Mr} = k5 \left( \frac{\sigma_{\text{conf}}}{\sigma_0} \right)^{k6} \left( 1 - k7 \left( \frac{\sigma_1}{\sigma_{1,f}} \right)^{k8} \right)$$  \hspace{1cm} (6.9)

where:

- $\sigma_1,f$: $\sigma_1$ at failure according to Mohr-Coulomb [kPa], see equation 6.2
- $\sigma_0$: reference stress $= 1$ kPa
- $k5$: model parameter [MPa]
- $k6, k7, k8$: model parameters [-]

The first term in equation 6.9 describes the increase of Mr with an increasing $\sigma_{\text{conf}}$. This dependency of Mr on $\sigma_{\text{conf}}$ or $\sigma_3$ was earlier found by Monismith et al. (34). The second term, describing the decrease of Mr with increasing $\sigma_1$, is added to the Mr-$\sigma_3$ model to obtain a model that describes the measured data much better than the Mr-$\Theta$ model does, see figure 6.10. Similar plots obtained for the other sands are given in appendix 6.4.

Values for the parameters $k5$ to $k8$ are given in table 6.12. By comparing the $r^2$-values given in table 6.12 with those given in table 6.11 it can be concluded that the new model is superior to the Mr-$\Theta$ model in describing the Mr data for sands.
**fig 6.10** The $Mr-\sigma_{conf}$ model and the measured $Mr$-values for the Zaanweg sub-base sand.

<table>
<thead>
<tr>
<th>sand</th>
<th>$k_5$ [MPa]</th>
<th>$k_6$ [-]</th>
<th>$k_7$ [-]</th>
<th>$k_8$ [-]</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>45.95</td>
<td>0.52</td>
<td>0.36</td>
<td>7.05</td>
<td>0.96</td>
</tr>
<tr>
<td>Weiver</td>
<td>38.40</td>
<td>0.60</td>
<td>0.89</td>
<td>5.79</td>
<td>0.98</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>44.96</td>
<td>0.52</td>
<td>0.28</td>
<td>5.83</td>
<td>0.98</td>
</tr>
<tr>
<td>Max Havelaarweg</td>
<td>38.64</td>
<td>0.68</td>
<td>0.42</td>
<td>4.07</td>
<td>0.94</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>30.42</td>
<td>0.59</td>
<td>0.47</td>
<td>7.68</td>
<td>0.98</td>
</tr>
<tr>
<td>Cor Bruinweg</td>
<td>47.75</td>
<td>0.58</td>
<td>0.67</td>
<td>1.51</td>
<td>0.95</td>
</tr>
<tr>
<td>Pascatweg</td>
<td>34.43</td>
<td>0.69</td>
<td>0.37</td>
<td>4.97</td>
<td>0.97</td>
</tr>
<tr>
<td>Crusher sand</td>
<td>19.23</td>
<td>0.54</td>
<td>0.19</td>
<td>8.63</td>
<td>0.99</td>
</tr>
</tbody>
</table>

**table 6.12** Parameters found for the $Mr-\sigma_{conf}$ model on different sands.

For the base course materials the decrease of $Mr$ with increasing $\sigma_1$ was not observed. Therefore the $Mr-\Theta$ model was applied to describe the stress-dependent resilient behaviour of these materials, see figure 6.11 and table 6.13.
The $M_r$-$\Theta$ model and the measured data for the Pascalweg base material.

<table>
<thead>
<tr>
<th>base course material</th>
<th>$k_1$ [MPa]</th>
<th>$k_2$ [-]</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>32.13</td>
<td>0.43</td>
<td>0.42</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>61.55</td>
<td>0.34</td>
<td>0.84</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>20.17</td>
<td>0.54</td>
<td>0.79</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>28.96</td>
<td>0.39</td>
<td>0.86</td>
</tr>
</tbody>
</table>

The fact that the triaxial test data for the base materials did not show a decrease of the $M_r$ with an increase of $\sigma_1$ is believed to be due to the rather low $\sigma_1/\sigma_{1,r}$-ratios at which the base materials were tested. The sands have been tested up to high $\sigma_1/\sigma_{1,r}$-ratios. This was done since calculations showed that the stress-ratios that can be expected in the sand layers of a concrete block pavement are very high and might even indicate failure. The base course materials are tested at fixed (lower) stress-levels since calculations showed that the stresses that occur in the base layers of concrete block pavements lead to much smaller $\sigma_1/\sigma_{1,r}$-ratios.

It is expected that the decrease of $M_r$ with increasing $\sigma_1$ will also be measured for the base materials if tests are performed up to higher $\sigma_1/\sigma_{1,r}$-ratios. In this research the goal of the tests was to get information about the
stress dependent behaviour within the range of stresses that are likely to occur in concrete block pavements. This implies that no tests were performed on base course materials at higher $\sigma_1/\sigma_{1,r}$-ratios.

Since not only the resilient axial strain but also the resilient radial strain was measured, not only the Mr modulus, but also the Poisson's ratio could be calculated.

The obtained data showed that the Poisson's ratio, $\nu$, is stress dependent and increases with the cyclic stress, $\sigma_{\text{cyc}}$. For the data obtained on the sands five models for explaining the stress dependency of $\nu$ were analyzed, in order to determine the best model using regression analysis. These models are presented in the equations 6.10 to 6.14.

\[
v = \nu_1 \frac{\sigma_1}{\sigma_3} + \nu_2 \tag{6.10}
\]

\[
v = \nu_3 \left( \frac{\sigma_1}{\sigma_3} \right)^{\nu_4} \tag{6.11}
\]

\[
v = \nu_5 \frac{\sigma_1}{\sigma_{1,r}} + \nu_6 \tag{6.12}
\]

\[
v = \nu_7 \left( \frac{\sigma_1}{\sigma_{1,f}} \right)^{\nu_8} \tag{6.13}
\]

\[
v = \nu_9 + \nu_{10} \left( \frac{\sigma_1}{\sigma_{1,f}} \right)^{\nu_{11}} \tag{6.14}
\]

where:
\[
\begin{align*}
\nu: & \quad \text{Poisson's ratio [-]} \\
\nu_1 \ldots \nu_{11}: & \quad \text{model parameter [-]}
\end{align*}
\]

As shown by table 6.14 the $r^2$-values found for the models explaining the stress dependency of $\nu$ are not as high as the values found for explaining the resilient modulus. The best results are obtained when $\nu$ is explained by the $\sigma_1/\sigma_{1,f}$-ratio (equations 6.12, 6.13 and 6.14). Explaining $\nu$ by the $\sigma_1/\sigma_3$-ratio (equations 6.10 and 6.11) leads to very poor results. The model parameters found for the equations 6.10 and 6.11 are not given in this report since the fit
between these models and the data was considered to be too poor to give any reliable information. For the three other tested models the results are given in the tables 6.15, 6.16 and 6.17.

<table>
<thead>
<tr>
<th>Used model</th>
<th>mean ( r^2 ) based on eight sands</th>
</tr>
</thead>
<tbody>
<tr>
<td>equation no. 6.10</td>
<td>0.194</td>
</tr>
<tr>
<td>equation no. 6.11</td>
<td>0.207</td>
</tr>
<tr>
<td>equation no. 6.12</td>
<td>0.664</td>
</tr>
<tr>
<td>equation no. 6.13</td>
<td>0.691</td>
</tr>
<tr>
<td>equation no. 6.14</td>
<td>0.733</td>
</tr>
</tbody>
</table>

**Table 6.14**  Mean \( r^2 \)-values based on regression analysis on the data for the sands.

<table>
<thead>
<tr>
<th>Sand</th>
<th>( \nu_5 )</th>
<th>( \nu_6 )</th>
<th>( r^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crusher sand</td>
<td>0.658</td>
<td>-0.051</td>
<td>0.953</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>0.512</td>
<td>-0.068</td>
<td>0.520</td>
</tr>
<tr>
<td>Baarweg</td>
<td>0.391</td>
<td>-0.057</td>
<td>0.811</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>0.546</td>
<td>-0.134</td>
<td>0.658</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>0.559</td>
<td>-0.249</td>
<td>0.548</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>0.476</td>
<td>-0.072</td>
<td>0.290</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>0.518</td>
<td>-0.201</td>
<td>0.679</td>
</tr>
<tr>
<td>Weiver</td>
<td>0.746</td>
<td>-0.203</td>
<td>0.856</td>
</tr>
</tbody>
</table>

**Table 6.15**  Results of the regression analysis of the measured \( v \) for equation 6.12.

<table>
<thead>
<tr>
<th>Sand</th>
<th>( \nu_7 )</th>
<th>( \nu_8 )</th>
<th>( r^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crusher sand</td>
<td>0.602</td>
<td>1.107</td>
<td>0.958</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>0.438</td>
<td>1.251</td>
<td>0.588</td>
</tr>
<tr>
<td>Baarweg</td>
<td>0.323</td>
<td>1.150</td>
<td>0.818</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>0.400</td>
<td>1.411</td>
<td>0.685</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>0.327</td>
<td>2.062</td>
<td>0.604</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>0.373</td>
<td>1.237</td>
<td>0.274</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>0.291</td>
<td>1.710</td>
<td>0.735</td>
</tr>
<tr>
<td>Weiver</td>
<td>0.547</td>
<td>1.579</td>
<td>0.868</td>
</tr>
</tbody>
</table>

**Table 6.16**  Results of the regression analysis of the measured \( v \) for equation 6.13

Material Testing 117
<table>
<thead>
<tr>
<th>sand</th>
<th>(\nu_9)</th>
<th>(\nu_{10})</th>
<th>(\nu_{11})</th>
<th>(r^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crusher sand</td>
<td>0.107</td>
<td>0.535</td>
<td>1.724</td>
<td>0.967</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>0.041</td>
<td>0.400</td>
<td>1.477</td>
<td>0.588</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>0.148</td>
<td>0.234</td>
<td>4.613</td>
<td>0.885</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>0.100</td>
<td>0.312</td>
<td>2.389</td>
<td>0.692</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>0.092</td>
<td>0.345</td>
<td>5.823</td>
<td>0.656</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>0.126</td>
<td>0.454</td>
<td>3.950</td>
<td>0.299</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>0.116</td>
<td>0.266</td>
<td>6.797</td>
<td>0.868</td>
</tr>
<tr>
<td>Weiver</td>
<td>0.165</td>
<td>0.578</td>
<td>4.225</td>
<td>0.907</td>
</tr>
</tbody>
</table>

*table 6.17  Results of the regression analysis of the measured \(\nu\) for equation 6.14.*

In figure 6.12 the stress dependency of \(\nu\) as measured for the Zaanweg sub-base sand is shown together with the relation predicted by equation 6.14.

![Graph showing stress dependent \(\nu\) as measured for the Zaanweg sand.](image)

*fig 6.12  The stress dependent \(\nu\) as measured for the Zaanweg sand.*

For the base course materials also a stress dependency of \(\nu\) was found. The \(\nu\)-values found for these materials also showed an increase with increasing \(\sigma_{cyl}/\sigma_s\)-ratio and \(\sigma_t/\sigma_{t,r}\)-ratio. The \(\nu\)-values measured for these materials are higher than the values measured for the sands.
For the base course materials the $\nu$-value measured at small stress ratios, i.e. a $\sigma_{\text{cyl}}/\sigma_{\tau}$-ratio of 2 to 3, was mostly about 0.4. With an increase of the stress ratio the $\nu$-value increases rapidly and becomes higher than 0.5. This implies that the $\nu$-value of a base course material is larger than 0.5 if the material is only slightly stressed.

No attempt has been done to describe the stress-dependency of the $\nu$ for the base course materials since the structural model discussed in chapter four can not handle $\nu$-values higher than 0.5, see also chapter 7.

6.7 PERMANENT DEFORMATION BEHAVIOUR UNDER A CYCLIC TRIAXIAL TEST

6.7.1 Test description

The cyclic triaxial test was also used to determine the permanent deformation behaviour of the sands and the base course materials. For testing the sands the small hydraulic set-up, discussed in appendix 6.3 and shown in figure a6.3.1 was used. For the base course materials the large set-up discussed in appendix 6.4 and presented in the figures a6.4.2 and a6.4.3 was used.

The permanent strain tests show a large resemblance with the resilient strain tests, see appendix 6.4. Again the sample is first subjected to the confining stress only. In this research all permanent strain tests are conducted at a $\sigma_{\text{conf}}$-level of 12 kPa. This stress-level is found at a depth of 0.6 m underneath the road surface, assuming that dead weight stresses are all-around and that the specific weight of the materials used is 2000 kg/m$^3$.

After applying $\sigma_{\text{conf}}$ first a small static stress and then an additional halfsine shaped cyclic stress is applied to the sample by the actuator in a force controlled mode.

During testing up to 1,000,000 load repetitions were applied to a sample. The cyclic stress in this test had a frequency of 5 Hz. The load pulse had a duration of 100 ms followed by a rest period of 100 ms. Using this frequency almost 56 hours were needed to apply the 1,000,000 load repetitions.

6.7.2 Test results

The results of the permanent strain triaxial tests again show stress-dependent behaviour. The higher the applied $\sigma_{\text{cyl}}$ the stronger the build-up of
permanent strain as is shown in the figures 6.13 to 6.16.

The outcome of an individual permanent strain cyclic triaxial test can be plotted on a log-natural scale as was done by Barksdale (35) who performed numerous tests in which 100,000 load repetitions are applied.

Sweere (22), who tested granular materials up to 1,000,000 load repetitions, however found that the log-natural approach was not able to cope with the strain development at a larger number of load repetitions. He found that a better description of permanent strain cyclic triaxial test results was obtained by a log-log approach.

Similar to what Sweere found for laboratory test results, the log-log approach was also used by Houben and Veverka (14, 15, 36) to describe the permanent strain development in granular materials in pavements under traffic.

In this research the development of permanent strain in the sands is described by a model that results in straight lines on a log-log scale, see equation 6.15.

\[
\varepsilon_{\text{perm}} = A \times \left( \frac{N}{1000} \right)^B
\]

(6.15)

where:
\[ \varepsilon_{\text{perm}}: \quad \text{permanent strain [-]} \]
\[ A, B: \quad \text{model parameters [-]} \]

To implement the measured permanent strain development in the computation of permanent strain development in a pavement structure the permanent strain in the applied materials has to be known as a function of both the number of load repetitions and the stresses in the material.

As can be seen from the figures 6.13 and 6.14, A and B are a function of the applied stresses. Equation 6.16 and 6.17 are used to describe A and B as a function of the \( \sigma_1 / \sigma_{1f} \) ratio, this resulted in a good description of the measured permanent strains in the sands, see figure 6.13 and 6.14 and tables 6.18 and 6.19.

\[
A = a_1 \left( \frac{\sigma_1}{\sigma_{1f}} \right)^{a_2}
\]

(6.16)

\[
B = b_1 \left( \frac{\sigma_1}{\sigma_{1f}} \right)^{b_2}
\]

(6.17)
where:
\( a_1, a_2, b_1, b_2 \): model parameters [-]
\( \sigma_{1,r} \): first principal stress at failure [kPa]
\( \sigma_1 \): first principal stress [kPa]

**Fig 6.13** Development of permanent axial strain in the Zaanweg sub-base sand.

<table>
<thead>
<tr>
<th>sand</th>
<th>( a_1 % [-] )</th>
<th>( a_2 [-] )</th>
<th>( b_1 [-] )</th>
<th>( b_2 [-] )</th>
<th>( r^2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>-5.01</td>
<td>16.40</td>
<td>0.30</td>
<td>20.07</td>
<td>0.85</td>
</tr>
<tr>
<td>Weiver</td>
<td>-2.72</td>
<td>6.64</td>
<td>0.27</td>
<td>6.05</td>
<td>0.99</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>-2.54</td>
<td>5.26</td>
<td>0.07</td>
<td>0.00</td>
<td>0.86</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>-4.97</td>
<td>4.96</td>
<td>0.31</td>
<td>13.27</td>
<td>0.82</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>-3.53</td>
<td>9.59</td>
<td>0.76</td>
<td>27.80</td>
<td>0.80</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>-5.19</td>
<td>5.93</td>
<td>0.14</td>
<td>1.13</td>
<td>0.88</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>-5.46</td>
<td>6.61</td>
<td>0.29</td>
<td>6.06</td>
<td>0.95</td>
</tr>
<tr>
<td>Crusher sand</td>
<td>-3.02</td>
<td>2.98</td>
<td>0.24</td>
<td>1.76</td>
<td>0.66</td>
</tr>
</tbody>
</table>

**Table 6.18** Parameters for the development of permanent axial strain in the sands.

In the figures 6.13 and 6.14 the regression model is represented by the continuous lines; the measured data are indicated by means of three characters. Similar plots obtained for the other sands are presented in appendix 6.5.

The test data obtained on the sands indicate that sands only show a strong development of permanent strains if they are loaded by a \( \sigma_{cycl} \) that brings the \( \sigma_1/\sigma_{1,r} \) ratio very close to unity.
Fig 6.14 Development of permanent radial strain in the Zaanweg sub-base sand.

<table>
<thead>
<tr>
<th>sand</th>
<th>a1% [-]</th>
<th>a2 [-]</th>
<th>b1 [-]</th>
<th>b2 [-]</th>
<th>r²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>5.98</td>
<td>22.63</td>
<td>0.34</td>
<td>20.93</td>
<td>0.70</td>
</tr>
<tr>
<td>Weiver</td>
<td>3.13</td>
<td>9.16</td>
<td>0.32</td>
<td>8.67</td>
<td>0.98</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>2.03</td>
<td>6.09</td>
<td>0.09</td>
<td>2.49</td>
<td>0.83</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>5.42</td>
<td>7.75</td>
<td>0.40</td>
<td>29.47</td>
<td>0.91</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>4.20</td>
<td>14.89</td>
<td>0.90</td>
<td>25.44</td>
<td>0.95</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>6.82</td>
<td>8.29</td>
<td>0.16</td>
<td>2.38</td>
<td>0.91</td>
</tr>
<tr>
<td>Pascaiweg</td>
<td>6.50</td>
<td>9.12</td>
<td>0.51</td>
<td>10.64</td>
<td>0.95</td>
</tr>
<tr>
<td>Crusher sand</td>
<td>3.51</td>
<td>5.08</td>
<td>0.26</td>
<td>2.289</td>
<td>0.68</td>
</tr>
</tbody>
</table>

Table 6.19 Parameters for the development of permanent radial strain in the sands.

Examples of the permanent strain development measured in one of the four base course materials are given in figures 6.15 and 6.16. It is clear that the behaviour observed for higher \(\sigma_{i}/\sigma_{C}\)-ratios can not be described by means of equation 6.15. Therefore a second term was added to equation 6.15. The equation that is now obtained, equation 6.18, is commonly used to describe creep curves measured on asphaltic mixes, for instance by Francken et al. (37).

The model parameters C and D are again stress-dependent. Equations similar to the equations 6.16 and 6.17 are used to take this stress-dependency into account, see equations 6.19 and 6.20.
\[ \varepsilon_{\text{perm}} = A \times \left( \frac{N}{1000} \right)^B + C \left( e^{D \left( \frac{N}{1000} \right)} - 1 \right) \]  

(6.18)

where:
- \( \varepsilon_{\text{perm}} \): permanent strain [-]
- A, B, C, D: model parameters [-]

\[ C = c I \left( \frac{\sigma_1}{\sigma_{1f}} \right)^{c_2} \]  

(6.19)

\[ D = d I \left( \frac{\sigma_1}{\sigma_{1f}} \right)^{d_2} \]  

(6.20)

where:
- \( c_1, c_2, d_1, d_2 \): model parameters [-]
- \( \sigma_{1f} \): first principal stress at failure [kPa]
- \( \sigma_1 \): first principal stress [kPa]

**Fig. 6.15** Development of permanent axial strain in the Pascalweg base material.

In the figures 6.15 and 6.16 the results obtained for the Pascalweg base material are shown. Similar plots obtained for the other base course materials can be found in appendix 6.5. The tables 6.20 and 6.21 show the results that are obtained for the other base materials.
**Fig 6.16** Development of permanent radial strain in the Pascalweg base material.

<table>
<thead>
<tr>
<th>Base course material</th>
<th>$a1$ [%]</th>
<th>$a2$ [-]</th>
<th>$b1$ [-]</th>
<th>$b2$ [-]</th>
<th>$r^2$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>-492.95</td>
<td>10.51</td>
<td>14.62</td>
<td>4.95</td>
<td>0.864</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>-74.84</td>
<td>7.60</td>
<td>0.57</td>
<td>0.73</td>
<td>0.981</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>-2.66</td>
<td>7.50</td>
<td>119.00</td>
<td>10.00</td>
<td>0.972</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>-2.58</td>
<td>3.58</td>
<td>2.77</td>
<td>2.99</td>
<td>0.985</td>
</tr>
</tbody>
</table>

**Table 6.20** Parameters found for the development of permanent axial strain in the base course materials.
<table>
<thead>
<tr>
<th>Base course material</th>
<th>a1% [-]</th>
<th>a2 [-]</th>
<th>b1 [-]</th>
<th>b2 [-]</th>
<th>r² [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Allanstraat</td>
<td>270.20</td>
<td>9.83</td>
<td>21.29</td>
<td>5.54</td>
<td>0.790</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>M. Mavelaarweg</td>
<td>403.82</td>
<td>9.93</td>
<td>0.46</td>
<td>0.12</td>
<td>0.977</td>
</tr>
<tr>
<td></td>
<td>4.10</td>
<td>7.50</td>
<td>111.03</td>
<td>10.00</td>
<td></td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>11.15</td>
<td>5.41</td>
<td>1.25</td>
<td>2.15</td>
<td>0.982</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Pascalweg</td>
<td>3.48</td>
<td>4.19</td>
<td>227.36</td>
<td>5.27</td>
<td>0.964</td>
</tr>
<tr>
<td></td>
<td>1419.36</td>
<td>7.50</td>
<td>640.15</td>
<td>10.0</td>
<td></td>
</tr>
</tbody>
</table>

*Table 6.21 Parameters found for the development of permanent radial strain in the base course materials.*
Appendix 6.1
Volders and Verhoeven Sharpness

The Volders and Verhoeven Sharpness "VVS" is determined using a simple outflow test set-up, see figure a6.1.1. First of all 600 grams of the material to be tested is divided into five fractions: 0.063 to 0.125 mm, 0.125 to 0.25 mm, 0.25 to 0.5 mm, 0.5 to 1 mm and 1 to 2 mm. The sharpness of the material in each of these fractions is determined by testing two weight quantities of each fraction. These weight quantities depend on the weight of the fraction as shown by table a6.1.1.

![Diagram of Volders and Verhoeven test set-up.]

*Fig a6.1.1 Volders and Verhoeven test set-up.*

<table>
<thead>
<tr>
<th>Weight of the fraction, &quot;WF&quot; [g]</th>
<th>Weight quantity in first test [g]</th>
<th>Weight quantity in the second test [g]</th>
</tr>
</thead>
<tbody>
<tr>
<td>WF ≥ 120</td>
<td>± 120</td>
<td>± 80</td>
</tr>
<tr>
<td>80 ≤ WF &lt; 120</td>
<td>WF</td>
<td>± 80</td>
</tr>
<tr>
<td>60 ≤ WF &lt; 80</td>
<td>WF</td>
<td>± 60</td>
</tr>
<tr>
<td>WF ≤ 60</td>
<td>WF</td>
<td>WF</td>
</tr>
</tbody>
</table>

*Table a6.1.1 Weight quantities to be used for testing.*

First of all the material to be tested is placed in the container, see figure a6.1.1. After resetting the stopwatch, the slide gate at the bottom of the container is opened. The optical sensor reacts to the material going through the opening and signals the stopwatch which registers the outflow time. For each weight quantity this procedure is performed four times, the average
outflow time is then determined. Per fraction the average outflow time is determined for two weight quantities, as shown in table a6.1.1.

The outflow time, apart from the grain shape, also depends on the grain size and the specific gravity of the grain material. To take into account the influence of the grain size on the outflow time, the test set-up comes with three different outflow openings which can be mounted on the container. The fractions 0.063 to 0.5 mm are tested using the same outflow opening \((\phi = 6.4 \text{ mm})\). For the fraction 0.5 to 1 mm a somewhat larger outflow opening is used \((\phi = 7.9 \text{ mm})\). The third and largest outflow opening \((\phi = 13.5 \text{ mm})\) is mounted when the fraction 1 to 2 mm is tested.

The VVS of the five fractions is then determined by comparing the registered outflow times with the outflow times that were recorded by Volders and Verhoeven for two reference materials, i.e. glass pearls and crusher sand. For these reference materials Volders and Verhoeven determined, per fraction, the relation between outflow time and weight quantity by means of linear regression, see equation a6.1.1.

\[
OT_{sp} = a_{sp} \times W \times \left( \frac{SW_m}{SW_{av}} \right) + b_{sp} \tag{a6.1.1}
\]

where:

- \(OT_{sp}\): fictitious outflow time of the weight quantity in case of glass pearl shaped grains [s]
- \(SW_m\): specific weight of the grains of the material under consideration [g/cm³]
- \(SW_{av}\): average specific weight of sand grains = 2.65 [g/cm³]
- \(a_{sp}\): regression parameter found by Volders and Verhoeven [s/g]
- \(b_{sp}\): regression parameter found by Volders and Verhoeven [s]
- \(W\): weight of the material under consideration [g]

Equation a6.1.1 contains a correction for the specific gravity of the grain material. The equation results in a fictitious outflow time of the weight quantity for glass pearl shaped grains with a specific gravity that equals the specific gravity of the tested material, \(OT_{sp}\). Similar to this, the fictitious outflow time for reference crusher sand shaped grains can be determined, \(OT_{cs}\).

The values of the regression parameters for the glass pearls, \(a_{sp}\) and \(b_{sp}\), as well as the parameters for the reference crusher sand, \(a_{cs}\) and \(b_{cs}\), are presented in table a6.1.2.
<table>
<thead>
<tr>
<th>Fraction</th>
<th>$a_{gp}$ [s/g]</th>
<th>$b_{gp}$ [s]</th>
<th>$a_{cs}$ [s/g]</th>
<th>$b_{cs}$ [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 - 2 mm</td>
<td>0.023</td>
<td>0.29</td>
<td>0.052</td>
<td>0.272</td>
</tr>
<tr>
<td>0.5 - 1 mm</td>
<td>0.08</td>
<td>0.119</td>
<td>0.147</td>
<td>0.184</td>
</tr>
<tr>
<td>0.25 - 0.5 mm</td>
<td>0.108</td>
<td>0.096</td>
<td>0.179</td>
<td>0.241</td>
</tr>
<tr>
<td>0.125 - 0.25 mm</td>
<td>0.103</td>
<td>0.06</td>
<td>0.16</td>
<td>0.383</td>
</tr>
<tr>
<td>0.063 - 0.125 mm</td>
<td>0.101</td>
<td>0.016</td>
<td>0.19</td>
<td>-0.305</td>
</tr>
</tbody>
</table>

Table A6.1.2  Regression parameters for the determination of the fictitious outflow times.

On the basis of these fictitious outflow times the VVS per weight quantity, $\text{VVS}_w$, can now be determined:

$$\text{VVS}_w = 100\% \times \left( \frac{\text{OT}_w - \text{OT}_{gp}}{\text{OT}_{cs} - \text{OT}_{gp}} \right)$$  \hspace{1cm} (a6.1.2)

where:

$\text{OT}_w$: average outflow time of the weight quantity under consideration [s]

The VVS per fraction, $\text{VVS}_f$, equals the average of the VVS determined for the two weight quantities, $\text{VVS}_w$. In order to translate the $\text{VVS}_f$, determined for the five fractions, to the VVS of the total material a weight, $\text{U}_f$, is assigned to the five fractions, see equation a6.1.3:

$$U_f = \left( \frac{1}{\text{LL}_f} - \frac{1}{\text{UL}_f} \right) \times \left( \frac{4.343}{\log(\text{UL}_f) - \log(\text{LL}_f)} \right)$$  \hspace{1cm} (a6.1.3)

where:

$\text{LL}_f$: grain size at the lower limit of the fraction [mm]

$\text{UL}_f$: grain size at the upper limit of the fraction [mm]

From the sieve analysis the percentages of the material in the five fractions are known, $P_f$. The VVS of the material is now calculated using equation a6.1.4.

$$\text{VVS} = \frac{\sum_{f=1}^{5} (P_f \times U_f)}{\sum_{f=1}^{5} \left( \frac{P_f \times U_f}{\text{VVS}_f} \right)}$$  \hspace{1cm} (a6.1.4)
As discussed, the determination of the VVS requires the specific gravity of the grains of a material. Using a Pycnometer, figure a6.1.2, the total volume of a known (weighed) amount of material was exactly determined. Knowing the weight and the volume the specific gravity of the grains was determined.

*fig a6.1.2 Pycnometer.*
Appendix 6.2

Moisture-density and moisture-CBR relationship

In order to get insight into the moisture-density and the moisture-CBR (Californian Bearing Ratio) relationship of the sands involved in this research, CBR-tests were performed (31). This test requires a cylindrical mould and a collar which can be placed on a bottom-platen, see figure a6.2.1.

![Diagram of cylindrical mould and rammer.](image)

*fig a6.2.1* Cylindrical Proctor mould and rammer.

For testing a sand first of all a cylindrical metal disk is placed in the mould, see figure a6.2.1. Hereafter the mould is filled with the material to be tested. This is done in three equally thick layers. Each layer is compacted by 56 blows applied by dropping the rammer (figure a6.2.1) from 305 mm above the surface of the layer to be compacted. An apparatus is used that distributes the blows of the rammer evenly over the total surface of the sample and furthermore constantly maintains the desired dropping height.

After the three layers are compacted the collar is removed from the mould and the top surface of the material is smoothed using a metal edge. The weight of the compacted material in the mould is determined hereafter, given the dimensions of the mould this leads to the wet density, $\gamma_{\text{wet}}$.

The mould is then turned up side down and the bottom platen as well as the cylindrical metal disk (figure a2.1) are removed. The bottom platen is then placed on the other side of the mould.

Five metal load disks (2.27 kg each) are now placed on top of the material (these disks thus replace the metal disk which was on the bottom of
the mould during compaction). The five disks have a 54 mm diameter hole in their centre. The mould is now placed under an actuator which drives a cylindrical plunger with a 49.6 mm diameter into the material at a rate of 1.27 mm/min.

The force of the actuator is plotted against the penetration. The CBR value is determined on the basis of the actuator force at 2.54 mm (0.1 inch) penetration, $CBR_1$, and at 5.08 mm (0.2 inch) penetration, $CBR_2$, using the following equations.

$$CBR_1 = \frac{a}{1935 \times 7.005} \times 100\% = \frac{a}{135.6} \%$$

$$CBR_2 = \frac{b}{1935 \times 10.507} \times 100\% = \frac{b}{203.4} \%$$

(a6.2.1)

where:

a, b: actuator force at 2.54 and 5.08 mm penetration respectively [N]

1935: surface of the load area [mm$^2$]

7.005, 10.507: contact pressure on a standard sample of crushed rock at 2.54 and 5.08 mm penetration respectively [MPa]

In most cases $CBR_1$ is larger than $CBR_2$, in these cases the CBR value equals $CBR_1$. If $CBR_2$ exceeds $CBR_1$ then the test has to be repeated, if $CBR_2$ is again larger than $CBR_1$ then the CBR of the material equals $CBR_2$.

After the CBR test is performed the moisture content of the sample is determined and used to determine the dry density, $\gamma_{dry}$. A small amount of water is added to the material which is tested and a second test is performed at a somewhat higher moisture content. This is repeated until a sample starts loosing water during compaction. By plotting $\gamma_{dry}$, $\gamma_{wet}$ and the CBR against the moisture content insight is obtained into the moisture-density and moisture-CBR relationship of the sands.

For the base materials no CBR-tests were performed. Insight into the density moisture relationship for these materials was obtained by performing a somewhat different test called the one point Proctor test (31).

For this test first the fraction $> 31.5$ mm has to be determined. If this fraction is larger than 25% then no Proctor test can be performed. If this fraction is larger than 10% but smaller than 25% then the test is performed on the fraction $< 31.5$ mm. If the fraction $> 31.5$ mm is smaller than 10% then the test can be performed on the fraction $< 45$ mm.
For this test the density of the grains of the excluded fraction is determined by drying the excluded grains and then weighing them, giving the dry weight, \( W_d \). Hereafter the grains are submerged in water for 24 hours after which they are weighed under water, \( W_u \). The grains are now taken from the water container. Water drops attached to their surfaces are removed using a moisturized cloth and the wet grains are now weighed above water, \( W_w \). The dry density of the grains, \( \gamma_{gr, dry} \) [kg/m\(^3\)] and the wet density of the grains, \( \gamma_{gr, wet} \) [kg/m\(^3\)], are now determined by equation a6.2.2.

\[
\gamma_{gr, dry} = 1000 \times \frac{W_d}{W_w - W_u}
\]

and

\[
\gamma_{gr, wet} = 1000 \times \frac{W_w}{W_w - W_u}
\]

where:

- \( W_d \): weight of the dried grains of the excluded fraction [g]
- \( W_u \): weight of the wet grains of the excluded fraction [g]
- \( W_w \): under water weight of the wet grains of the excluded fraction [g]

After \( \gamma_{gr, dry} \) and \( \gamma_{gr, wet} \) are determined the fraction of the material that is to be tested can be compacted in a mould. This mould is similar to the mould in figure a6.2.1, its height is however 116.4 mm. The mould is placed on the bottom platen and the collar is placed on top of it. The material is compacted in the mould in three layers with a height of about 40 mm. The first two layers are compacted by 56 blows applied by a falling rammer in exactly the same way as discussed earlier. The third layer is only compacted by 40 blows. Hereafter a metal top platen is placed on top of the sample. An additional 20 blows are now applied on this top platen.

Given the dimensions of the mould, the volume of the sample can now be determined by measuring the distance between the top edge of the collar and the surface of the top platen at four places equally distributed over the collar.

The total weight of the mould and the material is thereafter determined, the weight of the material is then determined by subtracting the weight of the mould form the total weight. From the sample volume and the sample weight the wet sample density can be determined, \( \gamma_{s, wet} \).

Hereafter the sample moisture content is determined and used to compute the dry sample density, \( \gamma_{s, dry} \). On the basis of the dry sample density, the dry grain density and the dry weight percentage of the grains not involved in the actual test, \( \text{PD}_{e} \), the corrected proctor density, \( \text{P}_{d} \), can be computed.

---

Appendix 6.2: Moisture-density and moisture-CBR relationship 133
\[ PD_c = \frac{100 \times \gamma_{s,dry} \times \gamma_{gr,dry}}{100 \times \gamma_{gr,dry} - P_{dry} \times (\gamma_{gr,dry} - \gamma_{s,dry})} \] (a6.2.3)

Using the same equation the corrected wet material density can be determined. In this case use is however made of the wet densities determined. The wet weight percentage of grains not involved in the actual test, \( P_{wet} \), is in this case to be used instead of \( P_{dry} \). \( P_{wet} \) is determined using the following equation.

\[ P_{wet} = \frac{100 \times P_{dry} \times \left( \frac{\gamma_{gr,wet}}{\gamma_{gr,dry}} \right)}{P_{dry} \times \left( \frac{\gamma_{gr,wet}}{\gamma_{gr,dry}} \right) + (100 - P_{dry}) \times \left( \frac{\gamma_{s,wet}}{\gamma_{s,dry}} \right)} \] (a6.2.4)

where:
- \( P_{dry}, P_{wet} \): mass percentage of the fraction excluded from the actual test for the wet and the dry material respectively [-]
- \( \gamma_{s,dry}, \gamma_{s,wet} \): dry and wet sample density respectively [kg/m³]
- \( \gamma_{gr,dry}, \gamma_{gr,wet} \): dry and wet density of the grains excluded from the test respectively [kg/m³]

If the corrected dry density and the corrected wet density of the total base course material are known the corrected moisture content can easily be computed on the basis of these two densities.

The Dutch specification (31) prescribes that this test on base course materials should be performed once. The moisture content of the material for this single test is determined by considering the not compacted material. Water should be added until all the grains are moisturized and a slightly plastic mixture is obtained.

For this research the test was however performed several times, using a mixture with an increasing moisture content. Again the test was ended when the sample loosed water during compaction.

Without further comment the results of the described tests performed on the eight sands and four base course materials are presented on the following pages.
fig a6.2.2  Moisture-density and moisture-CBR relationship, crusher sand.

fig a6.2.3  Moisture-density and moisture-CBR relationship, M. Havelaar sub-base sand.

fig a6.2.4  Moisture-density and moisture-CBR relationship, Baarsweg sub-base sand.

Appendix 6.2: Moisture-density and moisture-CBR relationship
**fig a6.2.5** Moisture-density and moisture-CBR relationship, Pascalweg sub-base sand.

**fig a6.2.6** Moisture-density and moisture-CBR relationship, Zaanweg sub-base sand.

**fig a6.2.7** Moisture-density and moisture-CBR relationship, C. Bruynweg sub-base sand.

Appendix 6.2: Moisture-density and moisture-CBR relationship
**fig a6.2.8** Moisture-density and moisture-CBR relationship, Allenaat sub-base sand.

**fig a6.2.9** Moisture-density and moisture-CBR relationship, Weiver sub-base sand.

**fig a6.2.10** Moisture-density relationship, for the M. Havelaarweg base material, fraction < 45 mm.
fig a6.2.11 Moisture-density relationship, for the Allanstraat base material, fraction < 45 mm.

fig a6.2.12 Moisture-density relationship, for the C. Bruynweg base material, fraction < 31.5 mm.

fig a6.2.13 Moisture-density relationship, for the Pascalweg base material, fraction < 31.5 mm.
Appendix 6.3

Static triaxial test

The strength of a granular material, expressed in the first principal stress at failure, $\sigma_{1,fr}$, depends on the confining stress, $\sigma_{conf}$, applied on the material. To get insight into this stress-dependency static triaxial tests are performed at various confining stress levels.

In figure a6.3.1 the triaxial test set-up used to perform the static failure tests on the sands is presented. For testing a sand a cylindrical sample with a height of 200 mm and a diameter of 101.6 mm is prepared. These samples are made at optimum moisture content (the precise moisture content of the sample however can only be determined after testing).

For preparing a sample first a membrane is attached air tight to the bottom platen. Hereafter a split-mould is placed around the membrane and bottom platen. The moisturized sand is now compacted by hand in eight layers within the membrane in the split-mould. After compaction, the top platen is attached to the membrane, using underpressure the pore pressure in the sample is reduced and the split-mould is removed.

The sample is hereafter placed in the pressure cell. The pore pressure is equalized to the atmospheric pressure and the pressure in the cell is increased to the desired confining pressure level, $\sigma_{conf}$. The sand sample is now ready for testing.

![Diagram of triaxial test set-up]

**fig a6.3.1** The small hydraulic triaxial test set-up, sample size 200 mm x 101.6 mm.
Because of the maximum grain size of the base course materials a much larger triaxial test set-up was needed for testing the base course materials. This test set-up is schematically shown in figure a6.3.2.

![Diagram of triaxial test set-up]

**fig a6.3.2** *The large hydraulic triaxial test set-up, sample size 800 mm x 400 mm.*

In this larger test set-up samples with a diameter of 400 mm and a height of 800 mm are used. Besides the dimensions of the sample the main difference between the large test set-up and the small one is the way \( \sigma_{\text{conf}} \) is applied to the sample. In the small set-up this is done by over-pressurizing the pressure cell. In the large test set-up the confining pressure is applied by lowering the pore pressure in the sample (under-pressurizing). An upper and a lower vacuum gauge are used to monitor the \( \sigma_{\text{conf}} \) applied to the large sample, see figure a6.3.2.

The preparation of a sample for the large test set-up is very similar to the preparation of a sand sample. First material is moisturised up to a level closely to the optimum moisture content (experience learned that due to the underpressure, problems as a result of loss of water occur when a sample is prepared at the optimum moisture content). Again, first of all a membrane is placed over the bottom platen and connected air tight to it. Hereafter a split-mould is placed around the bottom platen and membrane. The material is now compacted within the membrane in the split-mould in eight layers. Each layer is compacted by hand, while each second layer is also subjected to 50,000 loads of about 50 kN applied by means of the actuator.
After compaction the top platen is placed and connected air tight to the membrane. The sample is connected to the underpressure, the split-mould is removed hereafter. Due to the violence needed to compact the sample the membrane will always have leaks. A second membrane is therefore put around the sample and connected air tight to the top and bottom platen. After \( \sigma_{\text{conf}} \) is adjusted to the desired level the sample is ready to be tested.

For performing a static failure triaxial test the actuator is put in a displacement controlled mode, with a constant loading speed of 1.27 mm/min. The actuator force and displacement are constantly recorded. Knowing the diameter of the sample the actuator force can be translated to the actuator stress, \( \sigma_{\text{act}} \), applied to the sample. With an increasing actuator displacement \( \sigma_{\text{act}} \) will at first also increase, see figure a6.3.3. After reaching a maximum, \( \sigma_{\text{act}} \) starts to decrease with a further increase of the actuator displacement. The first principal stress at failure, \( \sigma_{1,f} \), is now determined on the basis of the confining stress, \( \sigma_{\text{conf}} \), a small stress as a result of dead weight, \( \sigma_{\text{dw}} \), and the maximum \( \sigma_{\text{act}} \), see figure a6.3.3.

![Typical static triaxial failure test result.](image)

As discussed in chapter 6 the combined results of a few static triaxial failure tests can be used to determine the \( c \) and \( \phi \) values of a material. On the following pages the results of the static triaxial failure tests are presented by figures in which also the failure criterion of Mohr-Coulomb is shown.
**fig a6.3.4** Static failure triaxial test results and the Mohr Coulomb failure criterion for the crusher sand.

**fig a6.3.5** Static failure triaxial test results and the Mohr Coulomb failure criterion for the M. Havelaarweg sub-base sand.

**fig a6.3.6** Static failure triaxial test results and the Mohr Coulomb failure criterion for the Baarsweg sub-base sand.
**Fig a6.3.7** Static failure triaxial test results and the Mohr Coulomb failure criterion for the Pascalweg sub-base sand.

**Fig a6.3.8** Static failure triaxial test results and the Mohr Coulomb failure criterion for the Zaanweg sub-base sand.

**Fig a6.3.9** Static failure triaxial test results and the Mohr Coulomb failure criterion for the C. Bruynweg sub-base sand.
**fig a6.3.10** Static failure triaxial test results and the Mohr Coulomb failure criterion for the Allanstraat sub-base sand.

**fig a6.3.11** Static failure triaxial test results and the Mohr Coulomb failure criterion for the Wever sub-base sand.

**fig a6.3.12** Static failure triaxial test results and the Mohr Coulomb failure criterion for the M. Havelaarweg base material.
fig a6.3.13 Static failure triaxial test results and the Mohr Coulomb failure criterion for the Pascalweg base material.

fig a6.3.14 Static failure triaxial test results and the Mohr Coulomb failure criterion for the Allanstraat base material.

fig a6.3.15 Static failure triaxial test results and the Mohr Coulomb failure criterion for the C. Bruynweg base material.
Appendix 6.4

Resilient strain cyclic triaxial test

For testing the resilient behaviour of the granular materials again use was made of triaxial test set-ups. For testing the sands the UTM (Universal Testing Machine) test set-up with a pneumatic actuator was used, see figure a6.4.1.

This test set-up hardly differs from the small hydraulic triaxial set-up discussed in appendix 6.3. Again the specimen size is 200 mm x 101.6 mm and \( a_{\text{conf}} \) is again applied by placing the sample in a pressure cell. In this test set-up "on sample" radial strain transducers are used to measure the horizontal sample deformation. An external LVDT is used to measure the axial sample deformation.

In order to have the least possible error the measured axial strains are corrected for deformation in the equipment. Here to a massive steel sample was tested. In this test it was assumed that no axial sample deformation occurred. All the measured deformation thus was considered to be a result of equipment deformation. The correction now consists of subtracting the axial strains measured on the steel sample from the axial strains measured on the sand samples.

![Diagram](image)

**fig a6.4.1** The UTM triaxial test set-up.

For testing the base course materials again the large hydraulic 400 mm x 800 mm triaxial test set-up is used. For measuring the sample deformation the
sample is instrumented as shown in figure a6.4.2. Two rings are mounted around the sample. On these rings proximity transducers are placed to measure the radial strain. Between the rings four vertical LVDT’s are placed which monitor the axial strain. A top-view of one of the rings is given in figure a6.4.3.

\[\text{fig a6.4.2 Instrumentation of the large triaxial sample.}\]

\[\text{fig a6.4.3 Top-view of one of the instrumented rings mounted on the base material sample.}\]

For testing the sands as well as the base course materials the sample preparation equals the sample preparation discussed in appendix 6.3.

During the resilient testing of a material the stresses presented in figure a6.4.4 are applied to the sample. The first stress applied is the constant...
confining pressure, $\sigma_{\text{conf}}$. If this pressure is stable and shows the desired level the actuator static stress is applied, $\sigma_{\text{stat}}$. This static stress is only a limited stress to ensure constant contact between the actuator piston and the top platen during dynamic testing. Hereafter the much larger dynamic halflsine shaped actuator stress, $\sigma_{\text{cyl}}$, is applied to the sample. This cyclic stress is applied to the sample with a frequency of 1 Hz, see figure a6.4.4. During the measurement, both the stresses applied by the actuator, $\sigma_{\text{act}}$, and the sample deformation are recorded.

![Schematic of stresses and sample](image)

*fig a6.4.4 Stresses applied to a sample during resilient testing.*

To reduce the influence of initiation effects, the sand samples are subjected to 500 load cycles at a $\sigma_{\text{conf}}$-level of 12 kPa before starting the actual test. The cyclic stress, $\sigma_{\text{cyl}}$, applied in these 500 cycles depends on the results of the static failure test performed at $\sigma_{\text{conf}}$ of 12 kPa and is chosen such that $\sigma_1$ equals 70% of the measured $\sigma_{1,t}$, thus $\sigma_1/\sigma_{1,t} = 0.7$. After applying these 500 load cycles the sample is ready for testing.

For the sands the measurements are performed at five $\sigma_{\text{conf}}$-levels, i.e. 6, 12, 24, 48 and 96 kPa. The aim is to apply at least five levels of $\sigma_{\text{cyl}}$ at each level of $\sigma_{\text{conf}}$. Per level of $\sigma_{\text{cyl}}$ 50 load repetitions are applied on the sample. During these 50 cycles both the radial and the axial strains, $\varepsilon_{\text{rad}}$ and $\varepsilon_{\text{ax}}$ respectively, are measured and stored in a computer. The stresses that are applied to the sample are measured and also stored in the computer. On the basis of the last 20 cycles the mean value of the cyclic stress, $\sigma_{\text{cyl}}$, and the mean resilient strains, $\varepsilon_{r,\text{ax}}$ and $\varepsilon_{r,\text{rad}}$, are determined.

Informative computations performed before the start of the triaxial tests showed that the stresses within the sands in concrete block pavements are

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Appendix 6.4: Resilient strain cyclic triaxial test
very close or even larger than the stresses at failure. For this reason it was decided to perform resilient tests up-to stress ratios very close to failure, so that the measurement data will cover the range of stresses which develop in practice. For this reason for testing the sands the following test procedure is followed.

At low $\sigma_{\text{cyl}}/\sigma_{\text{conf}}$-ratios the radial and axial strains are almost completely resilient. At higher $\sigma_{\text{cyl}}/\sigma_{\text{conf}}$-ratios the radial and axial strains are no longer completely resilient. During the measurements the radial strain was constantly monitored. By the stepwise increase of $\sigma_{\text{cyl}}$ during the measurements performed at one constant $\sigma_{\text{conf}}$-level all of a sudden a $\sigma_{\text{cyl}}/\sigma_{\text{conf}}$-level is reached where permanent radial strain is collected, see figure a6.4.5. At this $\sigma_{\text{cyl}}/\sigma_{\text{conf}}$-level the last measurement at that particular $\sigma_{\text{conf}}$-level is performed. To limit the build-up of permanent strain less than 50 load cycles are applied during this last measurement at the $\sigma_{\text{conf}}$-level under consideration.

After this last measurement, the $\sigma_{\text{conf}}$-level is increased. A new set of measurements is now performed, again until a $\sigma_{\text{cyl}}$-level is reached where permanent radial strain develops.

![Diagram of load and radial strain over time](image)

*fig a6.4.5 Development of radial sample strain.*

The base materials are tested using a more straight forward procedure. The stresses applied to these base course materials are shown in table a6.4.1. Again the tests starts at the lowest level of both $\sigma_{\text{conf}}$ and $\sigma_{\text{cyl}}$. At each stress stage 100 load cycles are applied to the sample. During the last cycle the computer scans the signals of the load cell and the strain transducers and stores the measured signals.

After completing a test, another test is performed at the next $\sigma_{\text{cyl}}$-level. If the last test for a certain $\sigma_{\text{conf}}$-level is performed then $\sigma_{\text{conf}}$ is increased to the
next level. The next set of tests is now started, again beginning at the smallest level of $\sigma_{\text{cyc}}$.

For the base materials the tests are performed at two levels of $\sigma_{\text{sat}}$, i.e. 6 and 12 kPa.

<table>
<thead>
<tr>
<th>$\sigma_{\text{conf}}$ [kPa]</th>
<th>$\sigma_{\text{cyc}}/\sigma_{\text{conf}}$-ratio [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>2, 4, 6, 8, 10, 12, 14, 15, 16, 18, 19, 20, 21</td>
</tr>
<tr>
<td>12</td>
<td>2, 4, 6, 8, 10, 11, 12, 13, 14</td>
</tr>
<tr>
<td>24</td>
<td>2, 4, 6, 7, 8</td>
</tr>
<tr>
<td>36</td>
<td>2, 3, 4, 5, 6</td>
</tr>
<tr>
<td>48</td>
<td>2, 3, 4</td>
</tr>
</tbody>
</table>

Table A6.4.1 Stresses applied to the base course materials.

As discussed in chapter 6 the results of the resilient strain cyclic triaxial tests are used to determine the resilient modulus, $M_r$. For granular materials this modulus proves to be stress-dependent. On the next pages the measured $M_r$ as well as the models used to describe the measured $M_r$-values are presented for both the sands and the base materials.

For the tested sands also the measured stress-dependent Poisson’s ratio, $\nu$, is given on the next pages. Again the model that is used to describe this stress dependent $\nu$, discussed in chapter 6 (equation 6.14), is also plotted in the figures.
\textit{fig a6.4.6} Resilient modulus of the crusher sand.

\textit{fig a6.4.7} Poisson's ratio of the crusher sand.
fig a6.4.8 Resilient modulus of the Max Havelaarweg sub-base sand.

fig a6.4.9 Poisson's ratio of the Max Havelaarweg sub-base sand.
**Figure A6.4.10** Resilient modulus of the Baarsweg sub-base sand.

**Figure A6.4.11** Poisson’s ratio of the Baarsweg sub-base sand.
Fig a6.4.12 Resilient modulus of the Pascalweg sub-base sand.

Fig a6.4.13 Poisson's ratio of the Pascalweg sub-base sand.
**Fig a6.4.14** Resilient modulus of the Zaanweg sub-base sand.

**Fig a6.4.15** Poisson's ratio of the Zaanweg sub-base sand.
fig a6.4.16 Resilient modulus of the Cor Bruynweg sub-base sand.

fig a6.4.17 Poisson's ratio of the Cor Bruynweg sub-base sand.
**fig a6.4.18** Resilient modulus of the Allanstraat sub-base sand.

**fig a6.4.19** Poisson's ratio of the Allanstraat sub-base sand.
fig a6.4.20 Resilient modulus of the Weiver sub-base sand.

fig a6.4.21 Poisson's ratio of the Weiver sub-base sand.
fig a6.4.22 Resilient modulus of the Max Havelaarweg base material.

fig a6.4.23 Resilient modulus of the Pascalweg base material.
**fig a6.4.24** Resilient modulus of the Allanstraat base material.

**fig a6.4.25** Resilient modulus of the Cor Bruynweg base material.
Appendix 6.5
Permanent strain cyclic triaxial test

**Fig A6.5.1** Development of permanent axial strain in the crusher sand.

**Fig A6.5.2** Development of permanent radial strain in the crusher sand.
**fig a6.5.3** Development of permanent axial strain in the M. Havelaarweg sub-base sand.

**fig a6.5.4** Development of permanent radial strain in the M. Havelaarweg sub-base sand.
fig a6.5.5 Development of permanent axial strain in the Baarsweg sub-base sand.

fig a6.5.6 Development of permanent radial strain in the Baarsweg sub-base sand.
**fig a6.5.7** Development of permanent axial strain in the Pascalweg sub-base sand.

**fig a6.5.8** Development of permanent radial strain in the Pascalweg sub-base sand.
**fig a6.5.9** Development of permanent axial strain in the Zaanweg sub-base sand.

**fig a6.5.10** Development of permanent radial strain in the Zaanweg sub-base sand.
fig a6.5.11 Development of permanent axial strain in the C. Bruynweg sub-base sand.

fig a6.5.12 Development of permanent radial strain in the C. Bruynweg sub-base sand.
fig a6.5.13 Development of permanent axial strain in the Allanstraat sub-base sand.

fig a6.5.14 Development of permanent radial strain in the Allanstraat sub-base sand.
**fig a6.5.15** Development of permanent axial strain in the Weiver sub-base sand.

**fig a6.5.16** Development of permanent radial strain in the Weiver sub-base sand.
**fig a6.5.17** Development of permanent axial strain in the Max Havelaarweg base material.

**fig a6.5.18** Development of permanent radial strain in the Max Havelaarweg base material.
Fig a6.5.19 Development of permanent axial strain in the Pascalweg base material.

Fig a6.5.20 Development of permanent radial strain in the Pascalweg base material.
**fig a6.5.21** Development of permanent axial strain in the Allanstraat base material.

**fig a6.5.22** Development of permanent radial strain in the Allanstraat base material.
**Fig a6.5.23** Development of permanent axial strain in the Cor Bruynweg base material.

**Fig a6.5.24** Development of permanent radial strain in the Cor Bruynweg base material.
Calculation of permanent strain

7.1 INTRODUCTION

In the previous chapter the behaviour of the granular materials investigated in this research was discussed. It was shown that the materials show a stress-dependent behaviour. The main goal of this research, namely the explanation of the development of permanent deformation in concrete block pavements, requires a model in which both the stress-dependent resilient behaviour as well as the stress-dependent permanent strain development are considered.

The model to explain the development of permanent strain in concrete block pavements is extensively discussed in this chapter. It consists of two parts. In the first part, discussed in section 7.2, the stresses under a wheel load are computed taking into account the resilient behaviour of the applied materials.

The effects of the stress-dependent resilient behaviour of the granular materials on the resilient behaviour of the total concrete block pavement are discussed in the section 7.2.2. Hereafter some resilient calculations are discussed in which the stress-dependent behaviour of the materials applied in the pavement is taken into account, see section 7.2.3.

In section 7.3 the development of permanent strain throughout the substructure of concrete block pavements is discussed. In this section it is first explained how the results of the resilient calculation can be used to explain the development of permanent strain in the substructure (section 7.3.2). Hereafter some examples are discussed in section 7.3.3. These examples will give insight into the behaviour of a block pavement with a sand sub-base only and the behaviour of two pavements with both a base layer and a sand sub-base. One of these latter pavements has a 200 mm thick base, while the other pavement has a 300 mm thick base layer.

In the last section of this chapter some conclusions are given.
7.2 THE RESILIENT CALCULATION

7.2.1 Implementing the measured material behaviour

In chapter 6 the measured stress-dependent resilient behaviour of the granular materials involved in this research was discussed. Since the stresses that develop in the substructure heavily depend on the distribution of the stiffness throughout this substructure, the observed stress-dependent behaviour has to be implemented in the structural model discussed in chapter 5. Some assumptions are made to implement the models describing the stress-dependent behaviour of the granular materials into the structural model.

During the triaxial tests, \( \sigma_3 \) and \( \sigma_2 \) both equal the confining pressure, \( \sigma_{\text{conf}} \). The models describing the behaviour of the granular materials thus only apply for the case that \( \sigma_3 \) equals \( \sigma_2 \). Within the finite element calculations however \( \sigma_3 \) and \( \sigma_2 \) are not necessary equal to each other. It is therefore assumed that the behaviour of a granular material depends on the average value of \( \sigma_3 \) and \( \sigma_2 \).

It is stated that this assumption is hardly of practical influence on the computation results since \( \sigma_3 \) equals \( \sigma_2 \) at the central axis of the model. At greater distance from this axis of symmetry \( \sigma_3 \) and \( \sigma_2 \) differ. The computed stresses however become smaller with increasing distance to the central axis so that the assumption does hardly effect the computation results.

The structural model can not handle a Poisson's ratio, \( \nu \), equal or greater than 0.5. As discussed in chapter 6 such high strain ratios easily develop in base course materials. When only relatively low stresses are applied on these base course materials, strain ratio values greater than 0.5 were measured.

Since the finite element structural model is not capable in implementing \( \nu \)-values greater than 0.5, the \( \nu \) for the base materials is modelled by a constant value of 0.49.

For the sands the \( \nu \) is stress-dependent as discussed in chapter 6, see equation 6.14 and table 6.17. Only if the stresses in the sand combined with the model for \( \nu \) lead to a \( \nu \) larger than 0.49 this latter value is used. For five of the eight sands this only happens if the sand is overstressed \((\sigma_i/\sigma_{1,t}>1)\). For the other three sands the model explaining \( \nu \) as a function of the \( \sigma_i/\sigma_{1,t} \)-ratio only comes to Poisson's ratios larger than 0.49 if the \( \sigma_i/\sigma_{1,t} \)-ratio in the sand is very high (Crusher sand \( \sigma_i/\sigma_{1,t}>0.95 \), Weiver sub-base sand \( \sigma_i/\sigma_{1,t}>0.88 \), Cor Bruinweg sub-base sand \( \sigma_i/\sigma_{1,t}>0.88 \)).

The third assumption considers the horizontal stresses as a result of dead weight. In the analyses it is assumed that the horizontal dead weight stresses
in the substructure are equal to the vertical dead weight stresses.

The resilient calculation now becomes an iterative process. First of all a calculation is made on the basis of an estimated stiffness distribution. This computation of course gives the stresses in all elements of the model. For each element these calculated stresses are used to determine the material properties (Mr and ν in case of a sand) for the element. Of course the estimated properties do not equal the properties that follow from the actual calculated stresses in the element. A second calculation is thus needed.

In this second calculation the properties of each element are determined on the basis of the following equations 7.1 and 7.2.

\[
Mr_{e,j} = \frac{(r1 \times Mr_{e,j-1} + Mr_{e,o})}{(rI+1)} \quad (7.1)
\]

\[
ν_{e,j} = \frac{(r2 \times ν_{e,j-1} + ν_{e,o})}{(r2+1)} \quad (7.2)
\]

where:

- \(Mr_{e,j}\): Mr of element no. "e" during the next computation [MPa]
- \(ν_{e,j}\): ν of element no. "e" during the next computation [-]
- \(Mr_{e,j-1}\): Mr of element no. "e" during the previous computation [MPa]
- \(ν_{e,j-1}\): ν of element no. "e" during the previous computation [-]
- \(Mr_{e,o}\): Mr of element no. "e" based on the stresses computed during the previous computation [MPa]
- \(ν_{e,o}\): ν of element no. "e" based on the stresses computed during the previous computation [-]
- \(r1, r2\): parameters controlling the iteration process [-]

This iterative process is stopped when the difference between the properties used to compute stresses and the properties that follow from the stresses is less than 5% in all elements. In practice this implies that the error is far less than 5% in most of the elements in the finite element model.

To show the effects of implementing the stress-dependent behaviour of the granular layers in the substructure some computations are discussed in the next section.
7.2.2 Effects of stress-dependent resilient behaviour

In this paragraph the effects of the modelling of the resilient behaviour of the sands on the calculated stresses are discussed. Hereto three calculations are performed. These calculations all refer to a concrete block pavement with only a 1000 mm sand sub-base placed over a subgrade (E=60 MPa, ν=0.45). The thickness of the blocks is 80 mm (E=40,000 MPa, ν=0.25). The load is a 50 kN wheel load with a radius of the contact area of 150 mm.

In the first calculation the sand in the sub-base is modelled by a constant Young’s modulus of 100 MPa combined with a Poisson’s ratio of 0.35 (very commonly used values for sands in the Netherlands). In the second calculation the Mr-Θ model is applied in combination with a constant ν-value of 0.35. In the third calculation the Mr-σ₁-σ₃ model is used in combination with a stress-dependent ν. The model parameters used in the last two calculations refer to the Zaanweg sub-base sand.

In figure 7.1 the deflection bowls calculated using the three material models are given. As shown by this figure, only minor differences in the calculated deflection bowls are found.

![Deflection bowls calculated using three models for the resilient behaviour of the sub-base sand.](image)

As explained in the previous chapter the development of permanent strain in the materials used in the substructure of a pavement heavily depends on the stresses that develop in this substructure. Especially the \( \sigma_i/\sigma_{1,r} \) ratio is important. If this ratio is smaller than one then the stresses do not lead to failure of the material, when this ratio is however larger than one the stresses in the material will lead to shear failure.

In figure 7.2 the calculated \( \sigma_i/\sigma_{1,r} \) ratio in the centre of the model is
plotted against the depth. As shown by this figure major differences are now found between the three material models compared here.

The Mr-O model leads to a maximum $\sigma_1/\sigma_{1,r}$-ratio of about 4.6, and it is thus concluded that the discussed concrete block pavement will fail under 50 kN wheel loads. The same conclusion is drawn when one considers the results obtained for the case of a constant Young’s modulus. In this case the maximum $\sigma_1/\sigma_{1,r}$-ratio in the centre of the model equals about 1.3.

In The Netherlands however most concrete block pavements without a base layer show development of rutting, but shear failure of such concrete block pavements hardly occurs. The stresses calculated on the basis of the Mr-O model and the constant Young’s modulus model thus do not agree with reality.

When the Mr-$s_3$-$s_1$ model is used for the sub-base sand, the calculated $\sigma_1/\sigma_{1,r}$-ratios are smaller than one, indicating that the pavement will not fail under a 50 kN wheel load. The calculated $\sigma_1/\sigma_{1,r}$-ratios are however very close to one which implies that permanent strains will develop rapidly in the substructure when the concrete block pavement is subjected to 50 kN wheel loads. These conclusions are in agreement with the experience based on the behaviour of Dutch concrete block pavements with a sand sub-base only.

The differences in the stresses computed on the basis of the different material models can largely be explained by considering figure 7.3. This
figure shows for a particular sand the stress-strain relation as it develops using the three models.

![Material behaviour graph](image)

**fig 7.3** Calculated strain in Zaanweg sand with increasing $\sigma_1$ and constant $\sigma_2$ and $\sigma_3$ based on three material models.

The line plotted for the Mr-O model shows that this material becomes stiffer with an increase of $\sigma_1$. Since stiff areas in the structure attract stresses, a material modelled by the Mr-O model will attract stresses in a progressive way. The higher the stresses, the stiffer the material, the stronger it attracts stresses. This progressive attraction of stresses is in no way limited by material strength, resulting in large $\sigma_1/\sigma_{1,r}$-ratios.

The line plotted for the constant modulus model is a pure straight line indicating that the stiffness of the material does not depend on the stresses in the material. The material modelled in this way is thus not progressively attracting stresses. As a result much lower $\sigma_1/\sigma_{1,r}$-ratios are now computed.

The curve line is determined using the Mr-$\sigma_3$-$\sigma_1$ model. This line shows a decrease in stiffness with an increase in $\sigma_1$. The decrease in the stiffness is a result of the increase in the $\sigma_1/\sigma_{1,r}$-ratio due to the increase of $\sigma_1$ at a constant $\sigma_2$ and $\sigma_3$ level. As soon as a particular part in the substructure is relatively highly stressed (high $\sigma_1/\sigma_{1,r}$-ratio) it looses part of its stiffness and neighbouring parts will be more activated as a result of this (they are stiffer and thus attract stresses). All this means that a larger area is activated and smaller $\sigma_1/\sigma_{1,r}$-ratios are computed using the Mr-$\sigma_3$-$\sigma_1$ model then by using the other two models.
In the previous example the effects of $\nu$ are not considered. The stress-dependent $\nu$ however contributes to the bearing capacity of the concrete block pavement. If the $\sigma_1/\sigma_{1,r}$ ratio in the sand increases then $\nu$ also increases. As a result of the increasing $\nu$-value, the material in the centre of the model responds to over-stressing with increasing horizontal compressive stresses, locally increasing the strength of the material and so preventing local failure.

The explanation of the difference in the computed $\sigma_1/\sigma_{1,r}$ ratio also makes clear how three completely different material models can result in very similar deflection bowls. Using the Mr-Θ model the material in the centre of the model will show a large stiffness combined with large stresses. Using the Mr-$\sigma_3-\sigma_1$ model the material in the centre of the model will show smaller stresses in combination with a smaller stiffness. Since resilient strains heavily depend on the ratio between stress and stiffness, the resilient strains in both calculations are about equal, so that the deflections are about equal too.

The example as a result, clearly shows that the ability of back-calculating a deflection bowl does not tell much about the accuracy of the used material models and the reliability of the calculated stresses.

7.2.3 Results of the resilient calculations

7.2.3.1 Introduction

In this section the results of three resilient calculations are discussed. All three calculations refer to a concrete block pavement placed over a subgrade with an elastic modulus of 60 MPa. The $\nu$ of this subgrade is 0.45. The height of the substructure of the three pavements is 1000 mm over which 80 mm thick concrete blocks are placed.

One of the pavements only has a sand sub-base. The other two pavements have a substructure consisting of a 50 mm bedding sand layer placed over a base layer and a sand subbase. One of the two pavements with a base layer has a 200 mm thick base while the other one has a 300 mm thick base, see figure 7.4.

The resilient deformation characteristics of the bedding sand layer are assumed to be equal to those of the crusher sand. The sand sub-base is modelled by the Zaanweg sand and the base layer is modelled by the behaviour of the Pascalweg base material.

The joint spring stiffness of the normal springs interconnecting the modelled blocks is 5500 N/mm per mm modelled joint. The shear joint springs have a stiffness of 500 N/mm mm'. Both types of springs are active in the case of joint compression only.
In this section first of all the resilient pavement structure deformation is discussed. Hereafter the development of wheel load stresses in the substructure is presented. Finally the development of the $\sigma_1/\sigma_{1r}$-ratio throughout the substructure is discussed. In all cases the load applied to the pavement is a 50 kN wheel load with a circular loading area having a radius of 150 mm.

7.2.3.2 Resilient deformation

Since the three concrete block pavements discussed here are all loaded by the same wheel load, differences in resilient deformation can only be explained by differences in the stiffness of the substructure.

Figures 7.5 to 7.7 give insight into the properties of the finite element model and the resilient deformation of the three concrete block pavements. In these figures only the resilient deformations of the upper central part of the finite element mesh under the described wheel load are plotted.

In figure 7.5 the deformation of the concrete block pavement with a sand sub-base only is given. The figure shows rotation and translation of the concrete blocks. Especially in the first modelled joint, 96 mm from the axis of symmetry, the effects of modelling the joints by dimensionless springs is shown. The first and second concrete block are shown to overlap each other. In case of compressive joint normal forces this is always the case in joints modelled by dimensionless springs, see section 4.4.
**fig 7.5** Calculated resilient deformations in the pavement with only a sand sub-base.

**fig 7.6** Calculated resilient deformations in the pavement with a 200 mm thick base layer.
Similar to figure 7.5, figure 7.6 gives insight into the deformed mesh of the concrete block pavement with a 200 mm base layer. By carefully observing the deformed mesh the base layer can be recognized as a relatively stiff layer (smaller deformations).

The same can be observed in figure 7.7, in which the deformed mesh of the concrete block pavement with a 300 mm base layer is presented.

![Graph showing deformed mesh](image)

*fig 7.7* Calculated resilient deformations in the pavement with a 300 mm thick base layer.

As a result of the base material being stiffer than the sand it is expected that the maximum deflection decreases with the thickness of the base. In figure 7.8 the deformed concrete block layers of the three pavements over the first 1056 mm are presented to show this.

The deflections at a distance of 1056 mm from the load centre hardly depend on the thickness of the base layer. In the three calculations the deflection at this point varies from 0.141 mm for the pavement without a base to 0.140 mm for the pavement with a 300 mm base layer.

As shown by figure 7.8 the deflections at the load centre do depend on the substructure design. At this point the deflections are 0.825 mm, 0.720 mm and 0.679 mm for the pavement without a base, with a 200 mm base and with a 300 mm base respectively.
7.2.3.3 *Stresses and stiffness*

It is clear that the deflections discussed in the previous section are a result of the stresses that develop in the substructure and subgrade. Depending on the stiffness, these stresses result in strains that cause the deflections. In this section the stiffness throughout the substructure for the three concrete block pavements is presented by means of contour plots.

These plots only give a general impression, they are based upon interpolation and extrapolation of the actual calculated values and thus do not completely agree with the outcome of the calculations.

In figure 7.9 the distribution of stiffness (Mr) is given for the concrete block pavement with a sand sub-base only. The plot shows that the stiffness directly underneath the loaded concrete block can locally exceed 150 MPa. With increasing distance from this point the stiffness decreases rapidly.

At a distance of about 1000 mm from the load centre the stiffness directly underneath the blocks is about 30 MPa. Since the wheel load stresses that develop directly underneath the blocks at such distance from the load centre are very small, this stiffness largely depends on the dead weight of the overlying concrete block layer.

With a growing depth the stiffness in the centre of the model also decreases. The influence of wheel load stresses becomes less and less important with increasing depth. Wheel load stresses however still affect the
stiffness at the bottom of the sand sub-base.

**fig 7.9** Stiffness [MPa] throughout the substructure of the pavement with a sand sub-base only.

**fig 7.10** Stiffness [MPa] throughout the substructure of the pavement with a 200 mm base layer.

In figure 7.10 the distribution of stiffness throughout the substructure of the pavement with a 200 mm base layer is presented. As shown by the plot the stiffness of the base layer is generally larger than the stiffness of the sand. In the base layer the largest stiffness is found in the upper part of the base
underneath the edge of the central block. At the centre of the model the stiffness of the base layer decreases with the depth. This decrease is strongest at a distance of about 130 mm from the centre of the model.

At a larger radial distance from the centre of the model the situation is the other way around. Now the largest stiffness is found at the bottom of the base while the smallest stiffnesses are found at the top of the base.

In figure 7.11 the distribution of stiffness throughout the substructure of the pavement with a 300 mm base layer is given. Again the largest stiffness of the base layer develops underneath the load at the upper part of this layer. At minor horizontal distances from the load centre the stiffness of the base layer decreases with the depth, at larger horizontal distances from the load centre the bottom part of the base layer is however the most stiff.

![Figure 7.11: Stiffness (MPa) throughout the substructure of the pavement with a 300 mm base layer.](image)

In figure 7.12 the distribution of the $\sigma_1/\sigma_{1,e}$-ratio throughout the substructure of the pavement with a sand sub-base only is presented. The figure shows that very high $\sigma_1/\sigma_{1,e}$-ratios are found at a depth of about 280 mm underneath the pavement surface. Very locally ratios larger than 0.98 develop. This indicates that at that depth the stresses in the sand are close to the failure level.

At first glance it might seem strange that the largest $\sigma_1/\sigma_{1,e}$-ratio is found at a depth of 280 mm underneath the road surface, while the largest stresses develop directly underneath the block layer. This is a direct result of the theory of Mohr-Coulomb (used to calculate $\sigma_{1,e}$) which indicates that the ratio
between stresses, and not the absolute value of the stresses, determines whether a granular material will fail or not.

**fig 7.12** $\sigma_1/\sigma_{1r}$ ratio throughout the substructure of the pavement with a sand sub-base only.

In figure 7.13 the distribution of the $\sigma_1/\sigma_{1r}$-ratio throughout the pavement with a 200 mm base layer is presented. The $\sigma_1/\sigma_{1r}$-ratios that develop in the sand sub-base hardly exceed 0.96 (contour line without label). The sand sub-base thus no longer shows critical $\sigma_1/\sigma_{1r}$-ratios.

**fig 7.13** $\sigma_1/\sigma_{1r}$ ratio throughout the substructure of the pavement with a 200 mm base layer.
In the base layer $\sigma_i/\sigma_{1,r}$-ratios up to somewhat more than 0.3 are calculated at the bottom of the base in the centre of the model. A very strong decrease in the $\sigma_i/\sigma_{1,r}$-ratio in the base layer is observed with increasing horizontal distance to the load centre.

Due to the limited height of the bedding sand layer no labels could be plotted along the contour lines in this layer. In order to give an impression of the $\sigma_i/\sigma_{1,r}$-ratios that are found in this layer it should be mentioned that the maximum $\sigma_i/\sigma_{1,r}$-ratio in this layer is 0.575. This maximum value develops directly underneath the second concrete block.

Figure 7.14 shows the $\sigma_i/\sigma_{1,r}$-ratios that develop in the substructure of the pavement with a 300 mm base layer. In this plot it is shown that the maximum $\sigma_i/\sigma_{1,r}$-ratio in the base layer is found in the centre of the model, about in the middle of the base. The $\sigma_i/\sigma_{1,r}$-ratios at this point are somewhat larger than 0.25. Again the $\sigma_i/\sigma_{1,r}$-ratio in the base shows a strong decrease with increasing radial distance from the load centre.

The $\sigma_i/\sigma_{1,r}$-ratios in the sand sub-base now remain smaller than about 0.92, implying that the sand is no longer stressed to critical stress levels.

![Graph showing $\sigma_i/\sigma_{1,r}$-ratios](image)

**Fig 7.14** $\sigma_i/\sigma_{1,r}$-ratio throughout the substructure of the pavement with a 300 mm base layer.

In the bedding sand layer of the pavement with a 300 mm base layer again only limited $\sigma_i/\sigma_{1,r}$-ratios are found. The maximum $\sigma_i/\sigma_{1,r}$-ratio found in this particular layer equals 0.564. Again this value is found directly underneath the second concrete block.
7.3 COMPUTATION OF PERMANENT VERTICAL TRAIN DEVELOPMENT

7.3.1 Introduction

In this section the computation of the development of permanent vertical strain is discussed. First of all it is explained how the results of the resilient calculations are translated to permanent strain development. Hereafter the assumptions made to take into account the effects of lateral wander are discussed. Finally the results of some computations are presented and discussed.

7.3.2 Converting stresses to permanent strain

In section 7.2.3 the results of three resilient computations are discussed. In that section the development of the $\sigma_d/\sigma_{1,r}$-ratio throughout the substructure of a concrete block pavement was shown. As discussed earlier this ratio is strongly related to the development of permanent strain, $\epsilon_{\text{perm}}$, in the various granular materials. By combining the results of the resilient calculation with the models explaining $\epsilon_{\text{perm}}$ as a function of $N$ and the $\sigma_d/\sigma_{1,r}$-ratio the permanent strain development can be determined as is explained hereafter.

From the finite element calculation the principal stresses $\sigma_1$, $\sigma_2$ and $\sigma_3$ are known. These stresses are based upon the vertical stress $\sigma_z$, the horizontal stresses $\sigma_{rr}$ and $\sigma_{\theta \theta}$ and the shear stress $\sigma_{rz}$. From these stresses the angle "$\alpha$" of the first principal stress to the vertical $z$ axis can be computed, figure 7.15 and equation 7.3.

![Diagram](image)

*Fig 7.15 The angle "$\alpha$" of the first principal stress to the vertical $z$-axis.*
\[ \alpha = \frac{1}{2} \arctan \left( \frac{2 \times \sigma_{rr}}{\sigma_{rr} - \sigma_{zz}} \right) \]  

(7.3)

For the computation of the permanent strains, the value of \( \alpha \) is of importance. Since the principal stresses and strains have the same direction, the angle "\( \alpha \)" also gives the direction of the first principal strain. Since the development of permanent strain in the various materials is known, it is now possible to translate the calculated \( \sigma_i/\sigma_{1,i} \)-ratio and \( \alpha \)-values to the permanent strain development, see equation 7.4 and 7.5.

\[ \epsilon_{p,z}^{i}(N) = \cos(\alpha^i) \epsilon_{p,1}^{i} \left( \frac{\sigma_1}{\sigma_{1,i}} \right)^{i} N \]  

(7.4)

\[ \epsilon_{p,r}^{i}(N) = \sin(\alpha^i) \epsilon_{p,1}^{i} \left( \frac{\sigma_1}{\sigma_{1,i}} \right)^{i} N \]  

(7.5)

where:
\( \epsilon_{p,z}^{i}(N) \): permanent vertical strain development in element \( i \) as a function of \( N \) [-].
\( \epsilon_{p,r}^{i}(N) \): permanent horizontal strain development in element \( i \) as a function of \( N \) [-].
\( \alpha^i \): angle "\( \alpha \)" in element \( i \) [-].
\( \epsilon_{p,1}^{i}(\sigma_i/\sigma_{1,i})^{i},N \): permanent axial=first principal strain development for the material represented in element \( i \) as a function of \( \sigma_i/\sigma_{1,i} \) and \( N \) [-].
\( \epsilon_{p,2/3}^{i}(\sigma_i/\sigma_{1,i})^{i},N \): permanent radial=second/third principal strain development for the material represented in element \( i \) as a function of \( \sigma_i/\sigma_{1,i} \) and \( N \) [-].
\( \sigma_i/\sigma_{1,i} \): \( \sigma_i/\sigma_{1,i} \)-ratio computed for element \( i \) [-].
\( N \): number of load repetitions [-].

In figure 7.16 the results of the translation from \( \sigma_i/\sigma_{1,i} \)-ratio and \( \alpha \) to permanent vertical strain is given for the concrete block pavement with a sand sub-base only. The plot shows the permanent vertical strain as calculated for \( N=1,000 \). The maximum vertical strain at \( N=1,000 \) is about 3% and develops in the centre of the model at a depth of about 280 mm underneath the surface of the concrete block pavement. Directly underneath the concrete block layer the permanent vertical strains remain smaller and they show a strong decrease with increasing distance to the load centre.
fig 7.16 Permanent vertical strain (in %) in the substructure of the pavement with a sand sub-base only after 1,000 load repetitions.

fig 7.17 Permanent vertical strain (in %) in the substructure of the pavement with a sand sub-base only after 10,000 load repetitions.

In figure 7.17 a similar plot is given for the situation after 10,000 load repetitions. By comparing this plot with figure 7.16 it is shown that the contour-lines plotted for permanent vertical strains less than about 0.5%
hardly differ. Apparently such minor permanent strains develop instantly, even at small stress-ratios. At larger N-values however no further strain development is observed.

The largest permanent vertical strain at N=10,000 is about 9%, whereas at N=1,000 this value was about 3%. This shows that permanent vertical strain is still accumulated at a larger number of load repetitions, in the areas in which higher $\sigma_1/\sigma_{1,\text{r}}$-ratios occur.

Figure 7.17 shows that the permanent strains directly underneath the block layer remain relatively small when compared to the maximum value that develops at a depth of about 280 mm. The maximum permanent vertical strain directly underneath the blocks is about 1% after 10,000 load repetitions.

Similar computations have been made for the pavements with a base layer. In figure 7.18 the permanent vertical strains after 1,000 load repetitions in the pavement with a 200 mm base layer are presented. As shown by this plot the permanent strains in the base layer are much smaller than the strains that develop in the sand sub-base.

In the sand sub-base of the pavement without a base layer the largest vertical strains developed at a depth of about 280 mm underneath the surface of the pavement. In the structure with a 200 mm base layer, the sand at this depth is replaced by base course material. As a result the large strains that developed in the sand no longer occur. Directly underneath the 200 mm base layer considerable permanent strain in the sand however still develops.

![Diagram showing permanent vertical strain](image)

**fig 7.18** Permanent vertical strain (in %) in the substructure of the pavement with a 200 mm base layer after 1,000 load repetitions.
fig 7.19  Permanent vertical strain (in %) in the substructure of the pavement with a 200 mm base layer after 10,000 load repetitions.

As is shown by the figures 7.19 and 7.20, which give the permanent vertical strain in the pavement with a 200 mm base layer after 10,000 and 100,000 load repetitions respectively, by far the largest permanent strains
develop in the sand sub-base. After 100,000 load repetitions the permanent vertical strain that has accumulated in the sand directly underneath the base layer under the centre of the load is very locally in excess of 15%.

The permanent strain in the base layer itself at that moment in N remains limited. As shown by the figures 7.18 to 7.20 the maximum vertical strain in the 200 mm base layer develops at the bottom of this layer. After 100,000 load repetitions very locally a maximum vertical strain of about 3% is found here.

In the bedding sand layer the permanent vertical strain also remains small. In this layer the maximum vertical strain does not even reach the value of 1%.

Figure 7.21 shows that the permanent vertical strains, that develop in the 300 mm thick base layer during the first 10,000 load repetitions, are small. Furthermore the largest permanent strains now develop in the middle of the base, whereas the 200 mm base layer showed the largest permanent strains at the bottom.

![Permanent vertical strain in the substructure of the pavement with a 300 mm base layer after 10,000 load repetitions.](image)

In figure 7.22 it is shown that even after 100,000 load repetitions the permanent strain in the 300 mm base layer remains small. In the sand subbase however larger permanent strains develop. In both the base layer and the sand sub-base the area in which permanent strains develop is only small,
indicating that this pavement will have a long design life. This becomes even more clear by considering the situation after 1,000,000 load repetitions, figure 7.23.

**Fig 7.22**  Permanent vertical strain (in %) in the substructure of the pavement with a 300 mm base layer after 100,000 load repetitions.

**Fig 7.23**  Permanent vertical strain (in %) in the substructure of the pavement with a 300 mm base layer after 1,000,000 load repetitions.
The permanent vertical strains in the 300 mm thick base layer remain smaller than 1% after 1,000,000 load repetitions. At the top of the sand sub-base the permanent vertical strains are much larger: very locally values of about 8% are found. In the bedding sand layer again only limited permanent vertical strains develop with a maximum of about 0.5%.

It is clear that the development of permanent vertical strain is strongly related to the development of permanent pavement surface deformation. Given the figures 7.18 to 7.23 it is thus concluded that the pavement with a 300 mm thick base layer performs much better than the pavement with a 200 mm thick base layer. This is especially shown by the fact that the largest permanent vertical strains in the 200 mm thick base layer develop at the bottom of the base which shows that the 200 mm base is relatively close to failure. Whereas the largest permanent strains in the 300 mm thick base layer are found in the middle of the base layer, indicating that the 300 mm base is very well capable in resisting the 50 kN wheel load.

The development of large permanent strains at the bottom of the 200 mm base layer indicates that combination of base layer stiffness and strength is not optimal. Due to the wheel load the horizontal stresses at the bottom of a stiff layer decrease strongly, and might even become tensile. If this layer is not thick enough this can result in larger \( \sigma_1/\sigma_{1,r} \)-ratios which will result in a stronger permanent strain development. In these cases the development of this permanent strain is not so much a result of a large vertical stress but much more of the drop in the horizontal confining stresses. Since the strength of unbound materials for a large part depends on the confining stresses it is clear that this is a dangerous situation.

In the 300 mm thick base layer the largest permanent vertical strains no longer develop at the bottom of the base. Apparently the combination of base layer stiffness and strength for this base layer is much better.

### 7.3.3 Permanent pavement surface deformation

The total vertical deformation at the pavement surface can be calculated from the permanent strains by multiplying the vertical strain with the height over which that strain occurs. However two factors should be considered, which are the permanent deformation of the subgrade under the sub-base and the effect of lateral wander of the wheel loads.

In this section it is first discussed how these factors are taken into account, then some calculation results are discussed.
7.3.3.1 Subgrade deformation and lateral wander

Up till now only the development of permanent strain (deformation) in the bedding sand layer, the base and the sub-base has been discussed. It will be obvious that the permanent strain development in the subgrade has to be considered too in order to come to a realistic surface deformation.

Since there is no laboratory data available for the various subgrades that exist in the Netherlands use is made of an empirical relation between resilient deformation and permanent deformation developed by Veverka (36). The findings of Veverka were also used for developing the D3 design method, see section 2.4. For soils Veverka gives the following equation.

\[ \varepsilon_p = \varepsilon_r \times [a + b \log(N)] \]  

(7.6)

where:
- \( \varepsilon_p, \varepsilon_r \): permanent strain and resilient strain respectively [-]
- a, b: model parameters [-]
- N: number of load repetitions [-]

Veverka found that the parameter "a" varies from -1.3 to 1.3 depending on the stiffness of the subgrade. The parameter "b" is constant and 0.7. In this research it is chosen to use an a-value of 0, so that the permanent deformation at N=0 equals 0.

The permanent strains in the subgrade can now be determined easily by considering the calculated resilient strains of this subgrade. The build-up of permanent vertical strains in all the layers underneath the undeformable concrete blocks can as a result be taken into account to compute the permanent surface deformation.

Real traffic will not travel exactly in one wheel track. A driver of a vehicle is simply not capable of steering its vehicle perfectly. He or she is constantly correcting the direction in which the vehicle is travelling. As a result, a transversal distribution of wheel loads will develop.

For motorway traffic this phenomenon was investigated by the Road and Hydraulic Engineering Division of the Dutch Ministry of Transport and Public Works (23). This research showed that the measured lateral wheel-shift patterns can very well be described by a normal distribution. The standard deviation of this normal distribution depends amongst other things heavily on the lane width, see table 7.1.
<table>
<thead>
<tr>
<th>Average traffic-lane width [m]</th>
<th>Standard deviation of lateral wander &quot;σ_{lw}&quot; [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.00</td>
<td>240</td>
</tr>
<tr>
<td>3.25</td>
<td>260</td>
</tr>
<tr>
<td>3.50</td>
<td>290</td>
</tr>
</tbody>
</table>

Table 7.1  Lateral spread of wheel loads as a function of the lane width (23).

It is clear that lateral wander will have its effects on the behaviour of any pavement and therefore it has to be taken into account. In this research this is done by considering three levels of lateral wander.

The first level shows a standard deviation, σ_{lw}, of 100 mm. Compared to table 7.1 this is a low value. But in built-up areas, where most concrete block pavements are found, the lane width can easily be smaller than 3.0 m as a result of parked cars, traffic guiding measures etc.

The next level considered is σ_{lw} = 200 mm. This level is seen as the lower limit for lateral wander on narrow city roads. The third level, σ_{lw} = 300 mm, is seen as the upper limit for lateral wander on wide city roads.

![Lateral distribution of wheel loads](image)

Figure 7.24  Lateral distribution of wheel loads.

In order to explain how the effects of lateral wander are taken into account, the situation given in figure 7.24 is considered. As shown by this figure it is assumed that the lateral distribution of wheel loads is very simple, there are only two wheel tracks. The wheel tracks are situated at a transversal distance of 300 mm from each other. The wheel track closest to the kerb,
wheel track 1, is used by 70% of the wheel loads. Wheel track 2 is only used by 30% of the wheel loads.

It is clear that both the wheels that travel over wheel track 1 and the wheels that travel over wheel track 2 will contribute to the development of permanent surface deformation, i.e. lateral unevenness or rutting. This implies that the permanent deformation in wheel track 2 is determined by the loads that travel over wheel track 1 as well as the loads that travel over wheel track 2.

In section 7.3.2 the development of permanent vertical strain in concrete block pavements was discussed. These permanent strains were a result of wheel load repetitions applied to the pavement on one particular spot, i.e. \( \sigma_{w0} = 0 \) mm. By integrating these permanent vertical strains over the height of the substructure and subgrade the permanent deformation at the pavement surface is obtained. In this way the development of the permanent pavement surface deformation can be computed for any distance from the load centre.

The effects of the wheel loads that are applied on wheel track 1 on the permanent surface deformation at wheel track 2 is determined by the permanent vertical strains that develop at a horizontal distance of 300 mm from the load centre, see figure 7.25.

![Fig 7.25 Effects of wheel loads on wheel track 1 on the development of surface deformation at wheel track 2.](image)

In order to calculate the development of the permanent surface deformation at wheel track 2 first of all the development of the pavement surface deformation due to the permanent vertical strains under the load centre is determined at \( \log(N) = 0 \) to 7 with a step of 0.1.

Similar to this, the development of the permanent surface deformation due to the permanent vertical strains 300 mm from the load centre is also determined. In figure 7.26 the results of this exercise are given.
The determination of the permanent surface deformation at wheel track 2 due to wheel loads on both wheel track 1 and wheel track 2 becomes a stepwise exercise based on the principle shown in figure 7.26. To determine the deformation under wheel path 2 after 100 load cycles, the surface deformation due to the first 100 cycles is simplified by straight lines, as indicated in figure 7.26. These lines show a constant increase of the surface deformation with \( N \), which equals the average increase over the first 100 load repetitions. This is important since we do not know the sequence in which wheel loads will travel either wheel track 1 or wheel track 2.

Since 30% of the wheel loads are applied on wheel track 2, the permanent surface deformation at wheel track 2 is for 30 out of 100 load repetitions determined by the line found for the permanent strains at 0 mm from the load centre. The surface deformation based on the permanent strain at 300 mm from the load centre determines the effects of the other 70 wheel load repetitions. These load repetitions are applied on wheel track 1 which is situated 300 mm from the wheel track at which the surface deformation is to be determined.

Figure 7.27 shows a principle plot. In this plot the straight lines that represent the surface deformation due to the first 100 load repetitions are again plotted. The steepest line is applied for 30 out of 100 repetitions while the other line is applied for 70 out of 100 repetitions. The permanent surface deformation at wheel track 2 due to 100 wheel loads travelling both wheel tracks with a 30-70 distribution as described is now found and indicated by a black dot.
Permanent vertical surface deformation at wheel track 2 as determined for wheels travelling in both wheel tracks.

In order to obtain the development of the surface deformation at wheel track 2 it is necessary to compute the surface deformation at that track for increasing N-values. In this example the next N-value for which the surface deformation is determined is 250 wheel load repetitions. Hereto first of all the computed surface deformation after 100 load repetitions is transferred to the plot first presented in figure 7.26, see figure 7.28. This level of permanent surface deformation is indicated by a fat horizontal line in both the figures 7.27 and 7.28.

The lines representing the development of surface deformation for the additional 150 repetitions.
The development of the surface deformation due to the additional 150 wheel load repetitions is again simplified by straight lines. Of course these lines should this time consider the effects of 150 load repetitions. The damage at wheel track 2 introduced by the first 100 wheel load repetitions now determines which 150 wheel load repetitions are to be considered, see figure 7.28. As is shown by this figure this implies that the effects of the wheel loads that travel wheel track 1 becomes less significant.

Of the additional 150 wheel load repetitions 30%, i.e. 45 wheel load repetitions, will take place over wheel track 2. The effects of these 45 wheel load repetitions on the surface deformation at wheel track 2 is determined by the steepest line found in figure 7.28. This line is based on the permanent strains as determined directly underneath the wheel load.

The other 105 additional wheel load repetitions take place at wheel track 1, which is 300 mm from the wheel track at which the surface deformation has to be determined. The effects of these 105 repetitions are determined by permanent vertical strains that develop 300 mm from the load centre, i.e. the less steep line found in figure 7.28.

Figure 7.29 gives the effects of the additional 150 wheel load repetitions on the permanent surface deformation at wheel track 2 due to wheel loads that travel both wheel tracks with the described 30-70 distribution. Again the permanent surface deformation found is indicated by a black dot.

![Figure 7.29](image.png)

**Figure 7.29** Surface deformation as a result of an additional 150 repetitions showing the described lateral distribution.

It is clear that the described algorithm can be repeated for increasing N, resulting in a complete plot of the development of surface deformation. On
the basis of the same algorithm the rut development in any point in the transversal direction can be determined. By repeating the algorithm for various points in the transversal direction the development of a complete transverse profile can thus be computed. The points at which the surface deformation is calculated do not have to be part of any wheel track, these points might very well not be travelled by any wheel loads at all.

Of course the same algorithm can also be applied for the determination of the effects of more complex transverse wheel load distributions. In this research the lateral wheel load distribution is described by a standard distribution. For the computation of the effects of lateral wander such a continuous distribution has to be translated to a discrete distribution. For this translation, steps with a width of 32 mm in the transversal direction are considered.

In figure 7.30 the discrete distribution, for the case that \( \sigma_{lw} = 200 \text{ mm} \), is shown. To come to the plotted discrete distribution it was assumed that no wheel loads will travel over a wheel track further than 3.25 times \( \sigma_{lw} \) from the centre of the track.

**Figure 7.30 Discrete transversal wheel load distribution obtained for \( \sigma_{lw} = 200 \text{ mm} \).**

For this research the surface deformation as a result of wheel loads showing lateral wander is calculated at the following N-values: 100, 250, 400, 550, 700, 850, 1000, 2500, 4000, 5500, 7000, 8500, 10000, 25000, 40000, 55000, 70000, 85000, 100000, 250000, 400000, 550000, 700000, 850000, 1000000, 2500000 and 4000000. Computations are stopped premature when the deformation in the centre of the track becomes larger than 35 mm.
As described the computation for taking into account the effects of lateral wander is based on considering the additional surface deformation introduced by an additional number of wheel loads. It might be clear that the surface deformation as discussed here is explained from permanent vertical strains determined on the basis of material behaviour. Within the calculation discussed here it is thus possible to consider also the additional permanent vertical strains in the substructure and subgrade introduced by an additional number of wheel load repetitions. Adding-up these additional strains also gives the development of vertical strain throughout the substructure as a result of wheel loads showing lateral wander.

7.3.3.2 **Calculated permanent surface deformation**

In this section some results of the rutting calculations are shown. The pavement structures which are considered here are the same structures as presented earlier in section 7.2.3. Again all the calculations refer to a 50 kN standard wheel load.

In figure 7.31 the permanent vertical strain is given as determined for the structure with a sand sub-base only after 1,000 and after 100,000 load repetitions with a $\sigma_{lw}$ of 200 mm.

![Diagram](image_url)

**fig 7.31** Permanent vertical strain (in %) in the pavement with a sand sub-base only after 1,000 and 100,000 load repetitions, $\sigma_{lw} = 200$ mm.

It can be seen in figure 7.31 that the development of ruts in the pavement with a sand sub-base only is mainly caused in the upper 500 mm of the sub-base (1% contour line). The area in which the maximum vertical...
permanent strains develop is situated at a depth of 280 mm below the surface of the pavement.

By comparing the figure 7.31 with the figures 7.16 and 7.17 it becomes clear that lateral wander leads to a much wider area in which permanent vertical strains develop. A further increase of $\sigma_{lw}$ results in a further widening of the area in which permanent strains develop, see figure 7.32.

![Figure 7.32](image)

**fig 7.32** Permanent vertical strain (in %) in the pavement with a sand sub-base only after 1,000 and 100,000 load repetitions, $\sigma_{lw}=300$ mm.

As a result of the increase of $\sigma_{lw}$ the permanent strain levels, computed for a certain number of load repetitions, decrease. The maximum vertical strain at $N=1,000$ is somewhat more than 2% in case of $\sigma_{lw}=200$ mm. When $\sigma_{lw}=300$ mm the maximum vertical permanent strain at $N=1,000$ decreases to less than 2%.

After 100,000 load repetitions the maximum permanent strain computed in case $\sigma_{lw}$ equals 300 mm is somewhat larger than 10%. In case $\sigma_{lw}=200$ mm this value is more than 13%. The depth at which the maximum permanent strains develop is not affected by the amount of lateral wander.

Figures 7.33 and 7.34 give the development of permanent strain in the substructure of the pavement with a 200 mm thick base layer. Both figures show that the permanent vertical strains in the base layer after 1,000 load repetitions are much smaller than the strains in the sand sub-base after 1,000 wheel load repetitions. Due to the positive effects of the base layer on the development of permanent vertical strains the situation after 1,000,000 50 kN wheel load repetitions can now be considered.
fig 7.33 Permanent vertical strain (in %) in the pavement with a 200 mm base layer after 1,000 and 1,000,000 load repetitions, $\sigma_{lw} = 200$ mm.

fig 7.34 Permanent vertical strain (in %) in the pavement with a 200 mm base layer after 1,000 and 1,000,000 load repetitions, $\sigma_{lw} = 300$ mm.

As is shown by figure 7.33 a permanent vertical strain of about 14% is found in the sand sub-base directly underneath the base layer after 1,000,000 wheel load repetitions with $\sigma_{lw} = 200$ mm. At the bottom of the base layer the permanent vertical strain, at that moment in N, locally is even larger and shows values larger than 16%.

As is shown by figure 7.34, an increasing amount of lateral wander has a positive effect on the development of permanent strain in the substructure of the pavement with a 200 mm base layer. Especially the permanent strains found at the bottom of the base layer after 1,000,000 wheel load repetitions
are much smaller. For $\sigma_{lw}=300$ mm the maximum strain found at the bottom of the base equals about 2.5%. The permanent vertical strains that develop in the sand sub-base directly under the base layer are far less effected by $\sigma_{lw}$. After 1,000,000 load repetitions the strains found here, decrease from about 14% to about 12% when $\sigma_{lw}$ increases from 200 mm to 300 mm.

By increasing the thickness of the base layer the development of larger permanent vertical strains at the bottom of the base can be prevented, see figures 7.35 and 7.36. In case of a 300 mm base layer the maximum permanent vertical strains no longer develop at the bottom of the base. In this case the largest permanent vertical strains are computed in the middle of the base. Even after 1,000,000 load repetitions these strains remain small, indicating that the pavement is stable and will thus show a long design life.

From this it can be concluded that in contrary to the 200 mm base layer, the 300 mm base layer is strong enough to resist stresses introduced by a 50 kN wheel load for a larger number of wheel load repetitions, even if the wheel loads only show a limited amount of lateral wander ($\sigma_{lw}=200$ mm).

![Diagram](image)

**fig 7.35** Permanent vertical strain (in %) in the pavement with a 300 mm base layer after 1,000 and 1,000,000 load repetitions, $\sigma_{lw}=200$ mm.
Permanent vertical strain (in %) in the pavement with a 300 mm base layer after 1,000 and 1,000,000 load repetitions, $\sigma_{w} = 300$ mm.

Permanent surface deformation, i.e. rutting, is of course a result of permanent strains that develop in the substructure and subgrade. In figure 7.37 the rut development in the pavement with a sand sub-base only is presented.

In the Netherlands the design criterion for concrete block pavements is the rut depth measured underneath a 1.2 m long straight edge. This relative rut depth is obtained by subtracting the permanent surface deformation calculated at 600 mm from the middle of the rut from the rut depth calculated in the middle of the rut (absolute rut depth).

\[ \sigma_{LW} = 200 \text{ mm} \quad \sigma_{LW} = 300 \text{ mm} \]

\[ \text{Lateral distance to middle of the rut [mm]} \]

<table>
<thead>
<tr>
<th>N</th>
<th>RD [mm]</th>
<th>N</th>
<th>RD [mm]</th>
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</thead>
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<tr>
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<td>100</td>
<td>1</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
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<tr>
<td>4</td>
<td>100,000</td>
<td>4</td>
<td>100,000</td>
</tr>
</tbody>
</table>

Rut development in the pavement with a sand sub-base only for $\sigma_{w} = 200$ mm and $\sigma_{w} = 300$ mm respectively.
Figure 7.37 shows that an increase of lateral wander reduces the absolute rut depth. The surface deformation at 600 mm is however increased if the amount of lateral wander increases. This implies that the positive effects of lateral wander on the development of the relative rut depth are expected to be larger than the effects of lateral wander on the absolute rut depth.

\[
\sigma_{lw} = 200 \text{ mm} \quad \quad \sigma_{lw} = 300 \text{ mm}
\]

Lateral distance to middle of the rut [mm]

\[\begin{array}{ccc}
N [\text{]} & RD [\text{mm}] & N [\text{]} & RD [\text{mm}] \\
1 & 100 & 3.27 & 1 & 100 & 2.74 \\
2 & 1,000 & 4.67 & 2 & 1,000 & 4.33 \\
3 & 10,000 & 6.43 & 3 & 10,000 & 6.00 \\
4 & 100,000 & 9.64 & 4 & 100,000 & 8.73 \\
5 & 1,000,000 & 22.32 & 5 & 1,000,000 & 15.82 \\
\end{array}\]

**fig 7.38** Rut development in the pavement with a 200 mm base layer for \(\sigma_{lw} = 200 \text{ mm}\) and \(\sigma_{lw} = 300 \text{ mm}\) respectively.

Figure 7.38 gives the rut development in the pavement with a 200 mm base layer. The permanent deformation that is now calculated is much smoother than the deformation calculated for the pavement with only a sand sub-base when only a limited number of wheel load passages occurred.

During the first 100,000 load repetitions the pavement with a 200 mm base shows much less rutting than the structure with only a sand sub-base. As discussed earlier larger permanent strains however develop at the bottom of the 200 mm base layer when the number of wheel load repetitions increases. This was especially found for the situation where the wheel loads only show a limited amount of lateral wander (\(\sigma_{lw} = 200 \text{ mm}\)).

As shown by figure 7.38 this local base layer failure has a strong effect on the rut that develops in the pavement. A steep rut will now develop, especially for the case that \(\sigma_{lw} = 200 \text{ mm}\). Given the small surface deformation at 600 mm the rut depth underneath a 1.2 m straight edge or relative rut depth will now hardly be smaller than the absolute rut depth.

As discussed earlier premature base layer failure can be prevented by applying a thicker base. In figure 7.39 the rut development for the pavement with a 300 mm base is presented. As is shown by this figure the ruts that will now develop again show a smooth transversal profile.

Even after 1,000,000 load repetitions the absolute rut depth does hardly exceed 5 mm. In case of \(\sigma_{lw} = 200 \text{ mm}\) the surface deformation at 600 mm is
about half the absolute rut depth. For $\sigma_{lw}=300$ mm the situation is even better. The relatively large surface deformation that develops at 600 mm indicates that the development of the relative rut depth is much more moderate than the already moderate development of the absolute rut depth.

$$\sigma_{lw} = 200 \text{ mm} \quad \sigma_{lw} = 300 \text{ mm}$$

![Diagram of rut development](image)

*Fig 7.39: Rut development in the pavement with a 300 mm base layer for $\sigma_{lw}=200$ mm and $\sigma_{lw}=300$ mm respectively.*

Both lateral wander as well as applying a thick enough base layer, in which no premature failure develops, reduce the development of the rut depth in the centre of the rut and enlarge the width of the rut profile. As a result their positive effect on the relative rut depth (rut depth underneath a 1.2 m long straight edge) is even stronger than their positive effect on the absolute rut depth.

When base layer failure occurs a more abrupt rut profile is computed. For such a rut the difference between the absolute and the relative rut depth is only minor. In the figures 7.40 and 7.41, which give the development of the absolute and relative rut depth respectively for the three pavements, these phenomena are clearly reflected.

The rut development in the pavement with only a sand sub-base is very strong. It is shown that the development of the absolute rut depth in this pavement is hardly effected by the amount of lateral wander. The development of the relative rut depth in this pavement is however strongly affected by the amount of lateral wander. Especially when $\sigma_{lw}$ becomes 300 mm, the development of the relative rut depth becomes much more moderate.

In The Netherlands a concrete block pavement is considered as being failed when the relative rut depth with a 30% probability of exceeding reaches 15 mm. Given this design criterion the rut development presented in figure 7.41 indicates that the design life of the pavement with only a sand sub-base

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Calculation of permanent strain development
in the case of $\sigma_{lw} = 300$ mm is about 4 times larger than the design life in the case of $\sigma_{lw} = 200$ mm.

**Fig. 7.40** The development of the absolute rut depth in three pavement structures.

**Fig. 7.41** The development of the relative rut depth in three pavement structures.

The pavement with a 200 mm base layer also shows an increase of the design life with an increase of the amount of lateral wander. For this pavement the effects of lateral wander on the design life are not so strong as for the pavement with a sand sub-base only.

The behaviour of the pavement with a 200 mm base layer furthermore
shows a rapid development of a certain relative rut depth which is reached after about 100,000 wheel load repetitions. After this the further development of rutting takes place at a relatively slow rate. Figure 7.41 shows that the rut depth at which the more moderate rut development starts, heavily depends on the amount of lateral wander. This implies that an increase of amount of lateral wander in the case of the pavement with a 200 mm base does not only prolong the design life but also has a positive effect on the rut depth during a larger part of this design life.

A similar conclusion can be made for the pavement with a 300 mm base layer. This pavement is hardly accumulating damage and thus shows by far the longest design life of the three pavements. As shown by figure 7.40 the development of the absolute rut depth in this pavement is hardly effected by the amount of lateral wander.

When the relative rut depth is however considered, a much stronger positive effect of lateral wander is found. Similar to the behaviour of the pavement with a 200 mm base it is now found that the rut depth level at which a moderate further accumulation of rut depth develops is strongly effected by the amount of lateral wander. Again this indicates that an increase in the amount of lateral wander not only prolongs the design life but also has a positive effect on the quality of the concrete block pavement during a larger part of this design life.

7.4 CONCLUSIONS

In this chapter it is shown that the finite element model in combination with stress-dependent material behaviour enable the determination of permanent strain development in the substructure of a concrete block pavement. Based on these permanent strains the development of permanent surface deformation of a concrete block pavement can be determined taking into account the effects of lateral wander. Rutting in concrete block pavements, with unbound layers only, is now explained.

The finite element model can of course be loaded by wheel loads with various load magnitudes. The radius of the wheel load can be varied too, which affects the contact pressure of the wheel load.

In this chapter the effects of the substructure design and the amount of lateral wander on the rut development were discussed. Of course the effects of the subgrade modulus on rutting can also be investigated.

To be short the models discussed in this chapter enable the determination of the rutting behaviour of concrete block pavements as a function of amongst
others: the wheel load magnitude, the contact pressure, the amount of lateral wander, the substructure design and the subgrade modulus. Given these options the models discussed thus enable the development of a design method.
8 Rutting performance model

8.1 INTRODUCTION

In the previous chapter some permanent strain and rut depth calculations are discussed. These calculations show the effects of a 50 kN ($r=150$ mm) wheel load on the rut development in three concrete block pavements. In reality however traffic consists of numerous wheel load magnitudes, with different contact pressures. As a result the design of a concrete block pavement can only be established if the behaviour of a block pavement is known under traffic that consists of various wheel loads.

In order to come to a design method numerous calculations similar to the ones discussed in the previous chapter have to be made. On the basis of these calculations the effects of various design parameters and traffic parameters on the development of the rut depth are quantified. By means of multiple regression the findings are then described by equations which can be used for concrete block pavement design.

8.2 PERFORMED CALCULATIONS

In order to be able to quantify the concrete block pavement behaviour with respect to rut development numerous calculation are made. Within these calculations the following traffic related parameters are varied:

- **Wheel load magnitude:**
  Of course the magnitude of the wheel loads that travel over a pavement have their effect on the development of ruts. To get insight into these effects the calculations considered three wheel load levels, i.e. 30 kN, 50 kN and 70 kN. These wheel loads referred to 60 kN, 100 kN and 140 kN axle loads respectively.

- **Wheel load contact area or contact pressure:**
  The 50 kN wheel load is introduced over a circular contact area with a radius of 150 mm so that the contact pressure becomes 707 kPa (7.07 bar). The effects of the 30 kN and 70 kN wheel loads are determined both for the situation of a constant contact area (radius 150 mm) and for a constant contact pressure of 707 kPa.
The 30 kN wheel loads are as a result introduced to the pavement over a circular contact area with a radius of 150 mm and 116.19 mm so that the contact pressure becomes 424 kPa and 707 kPa respectively. The 50 kN wheel load is only introduced over a circular area with a 150 mm radius so that the contact pressure equals 707 kPa. For the 70 kN wheel loads a contact area with a radius of 177.48 mm and 150 mm are considered, resulting in contact pressures of 707 kPa and 990 kPa respectively.

- Standard deviation of lateral wander:
  To get insight into the effects of the amount of lateral wander on the development of the rut depth the calculations consider three levels of $a_{lw}$: 100 mm, 200 mm and 300 mm.

The effects of the subgrade on the behaviour of concrete block pavements are investigated by considering three levels of subgrade moduli: 30 MPa, 60 MPa and 120 MPa.

Except for the subgrade modulus and the mentioned traffic parameters the development of rut depth is also effected by some pavement parameters or design parameters. In the calculations the following parameters are varied:

- Substructure height:
  Given the base layer thickness (see below), the height of the substructure determines the sub-base thickness. This sub-base thickness of course affects the rutting behaviour of a concrete block pavement. To get insight into these effects the following substructure thicknesses are considered: 500 mm, 1000 mm and 1500 mm, see figure 8.1 and appendix 8.1.

- Base layer thickness:
  As shown in chapter 7 the application of a base course layer has a major effect on the concrete block pavement rutting behaviour. To get insight into these effects three base layer thicknesses are considered: 0 mm (thus referring to a pavement without a base), 200 mm and 300 mm. **The pavements with a base layer all have a 50 mm thick bedding sand layer of crusher sand** between the block layer and the base. No calculations for pavements with a base layer are made for the situation that the substructure height is only 500 mm.
  To get insight into the behaviour of pavement structures with a very thick base layer some additional calculations are made considering base layers with a thickness of 450 mm and 750 mm. These additional calculations all consider concrete block pavements with a 1000 mm and
1500 mm thick substructure, see figure 8.1 and appendix 8.1

In figure 8.1 the substructure designs that follow from the above are presented. The substructure designs for the additional calculations, considering pavements with very thick base layers, are shown in the right part of the figure.

![Fig 8.1 Substructure designs for which rutting behaviour calculations are made.](image)

As shown earlier the research contains material behaviour of seven natural sands and four base materials. In the computations three natural sands are considered: a sand that results in good block pavement rutting behaviour (Weiver sand), a sand that results in average block pavement rutting behaviour (Zaanweg sand) and a sand that results in poor block pavement rutting behaviour (Pascalweg sand).

From the four base materials two materials are applied in the calculations; one of these base materials is better than average (Pascalweg base material) while the other is performing under average (M. Havelaarweg base material). In total this results in six material combinations.

Of course the number of parameters that are varied in the calculations are limited. The models discussed in the previous chapter can also be used to investigate the effects of, for instance, the concrete block thickness, the joint stiffnesses and the thickness of the bedding layer.

In these analyses the block thickness was kept at 80 mm. Also the stiffness of the normal and shear springs between the blocks, representing the joint stiffness, were kept constant, being 5500 N/mm per mm modelled joint for the joint normal spring and 500 N/mm mm' for the joint shear spring. In case of joint tensile stresses both springs are inactive (i.e. have a far lower stiffness).

Finally a constant bedding layer thickness of 50 mm was assumed.
In appendix 8.1 insight into the calculations made for this research are given. As will be explained in the next section the outcome of these calculations form the basis for the design method.

8.3 REGRESSION ANALYSES

8.3.1 Reduction of the calculation data

8.3.1.1 Introduction

In order to come to a design method it is necessary to know the behaviour of a concrete block pavement with regard to rutting as a function of the parameters that are varied in this research. This knowledge is obtained on the basis of calculations of the type discussed in chapter 7. The outcome of such a calculation consists of a series N-values, starting at 100 and increasing to 2,500,000, and for each N-value both the absolute rut depth and the rut depth underneath a 1.2 m straight edge is calculated.

For the analysis of the results of a large number of calculations of the type discussed in chapter 7, first of all a reduction of the calculation results is needed. This is done by describing the result of the individual rut depth computations by a model that largely equals the model used to describe the development of permanent strains in granular materials, see equation 6.18.

\[
RD = a_p \left( \frac{N}{1000} \right)^{b_p} + c_p \left( e^{d_p \left( \frac{N}{1000} \right)} - 1 \right) \quad (8.1)
\]

where:

RD: Rut depth [mm]

\(a_p, c_p, b_p, d_p\): model parameters [mm]

\(N\): number of wheel load repetitions [-]

The model given by equation 8.1 is applied to describe the development of both the absolute rut depth, \(RD_a\), and the rut depth underneath a 1.2 m straight edge or the relative rut depth, \(RD_r\). For both \(RD_a\) and \(RD_r\) the regression analysis is performed on a log-log scale and considers the calculated rut development from \(N=100\) to \(N=2,500,000\) or to the N-value at which \(RD_a\) becomes larger than 30 mm.

Within the regression analysis the value of \(d_p\) is arbitrarily fixed at a value of 0.2 [-/mm] times \(c_p\). This was necessary since the effects of the
second term in equation 8.1 can be eliminated by either a very small \( d_p \)-value or a very small \( c_p \)-value. As a result combinations of a huge \( d_p \)-value with a very small \( c_p \)-value (or vice versa) were found in the case that the second term was of no interest in the description of the rutting behaviour of a pavement. By relating \( d_p \) to \( c_p \) combinations of a huge \( d_p \)-value with a very small \( c_p \)-value can no longer occur.

In appendix 8.1 the input for the calculations that are performed are presented. As shown 99 calculations are performed to get insight into the behaviour of pavements with a sub-base only and 201 calculations are performed for concrete block pavements with a base layer.

8.3.1.2 Example of data reduction

In figure 8.2 an example is given of both the development of the calculated \( R_{D_u} \) and the development of this rut depth as explained by the regression model (equation 8.1), i.e. after data reduction. Figure 8.2 refers to three pavement structures discussed in chapter 7, see figure 7.40. All three pavement structures have a 1000 mm substructure over a 60 MPa subgrade. One of the pavements only has a Zaanweg sand sub-base while two of the pavements also have a base layer made from the Pascalweg base material. The thickness of the base, if any, is indicated in the figure as is the amount of lateral wander for which the rutting behaviour is determined. All pavements are loaded by 50 kN wheel loads with a 707 kPa contact pressure.

![Figure 8.2](image)

*Fig 8.2* The regression model and the calculation results for \( R_{D_u} \) for the three pavements discussed in chapter 7, see also figure 7.40.
<table>
<thead>
<tr>
<th>Base course thickness [mm]</th>
<th>$\sigma_{tw}$ [mm]</th>
<th>$a_p$ [mm]</th>
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<th>$c_p$ [mm]</th>
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<td>0.987</td>
</tr>
</tbody>
</table>

Table 8.1  Regression results for the development of $R_{Da}$ in the three pavements.

As indicated by figure 8.2 the regression model applied for the description of the calculation results shows a very good fit. The same conclusion is drawn on the basis of table 8.1 in which the actual results of the regression analyses performed to obtain the required data reduction are presented. As shown by this table the behaviour of the pavements without a base course differs from the behaviour of the pavements with a base course. The value of $c_p$ equals nil for the pavements without a base course, while the pavements with a base layer show a certain $c_p$-value. By considering the permanent deformation behaviour of the various materials used in the substructure it becomes clear that this difference in concrete block pavement behaviour is a reflection of the difference between the behaviour of the sands and the base materials, see chapter 6.

In figure 8.3 the results of the regression analyses on the development of $R_D$, are presented together with the actual calculation data.

In table 8.2 the results of the regression analyses performed for the development of $R_{DF}$ in the three pavements are presented. As shown by this table the correlations found for the development of $R_D$, are somewhat less than those found for the development of $R_{Da}$.
The regression model and the calculation results for RD, for the three pavements discussed in chapter 7, see also figure 7.41.

<table>
<thead>
<tr>
<th>Base course thickness [mm]</th>
<th>$\sigma_{tw}$ [mm]</th>
<th>$a_p$ [mm]</th>
<th>$b_p$ [-]</th>
<th>$c_p$ [mm]</th>
<th>$r^2$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>100</td>
<td>8.768</td>
<td>0.260</td>
<td>0.0000</td>
<td>0.988</td>
</tr>
<tr>
<td>0</td>
<td>200</td>
<td>6.111</td>
<td>0.268</td>
<td>0.0000</td>
<td>0.976</td>
</tr>
<tr>
<td>0</td>
<td>300</td>
<td>3.939</td>
<td>0.254</td>
<td>0.0000</td>
<td>0.959</td>
</tr>
<tr>
<td>200</td>
<td>100</td>
<td>4.338</td>
<td>0.170</td>
<td>0.0478</td>
<td>0.993</td>
</tr>
<tr>
<td>200</td>
<td>200</td>
<td>3.015</td>
<td>0.166</td>
<td>0.0294</td>
<td>0.984</td>
</tr>
<tr>
<td>200</td>
<td>300</td>
<td>1.925</td>
<td>0.144</td>
<td>0.0255</td>
<td>0.967</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>2.672</td>
<td>0.077</td>
<td>0.0083</td>
<td>0.996</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>1.885</td>
<td>0.058</td>
<td>0.0081</td>
<td>0.967</td>
</tr>
<tr>
<td>300</td>
<td>300</td>
<td>1.195</td>
<td>0.028</td>
<td>0.0077</td>
<td>0.790</td>
</tr>
</tbody>
</table>

Regression results for the development of RD, in the three pavements.

As shown by the tables 8.1 and 8.2 the regression analyses resulted in the values of $a_p$, $b_p$ and $c_p$ for a given pavement structure loaded by a given wheel load with a given $\sigma_{tw}$.
8.3.2 Description of the rutting parameters

In order to come to a rutting performance model the values of \( a_p \), \( b_p \) and \( c_p \), which describe a pavement’s rutting behaviour, should be known as a function of the wheel load magnitude "\( L \)" , the contact pressure "\( \sigma_c \)" , the standard deviation of lateral wander "\( \sigma_{lw} \)" , the thickness of the sub-base "\( t_b \)" , the thickness of the base course "\( t_b \)" and the subgrade modulus "\( E_0 \)". Hereto the \( a_p \), \( b_p \), and \( c_p \) values that follow from the individual calculations, see table 8.1 and 8.2, are described by regression models.

For this purpose first of all the wheel load "\( L \)" , the contact pressure "\( \sigma_c \)" and the standard deviation of lateral wander "\( \sigma_{lw} \)" are standardized.

\[
L_s = \frac{L}{L_r} \quad \text{and} \quad \sigma_{ls} = \frac{\sigma_{lw}}{\sigma_{lwr}} \quad \text{and} \quad \sigma_{cs} = \frac{\sigma_c}{\sigma_{cr}}
\]  (8.2)

where:

\( L_s \): Standardized wheel load [-]

\( L \): Wheel load [kN]

\( L_r \): Reference wheel load of 50 kN [kN]

\( \sigma_{ls} \): Standardized standard deviation of lateral wander [-]

\( \sigma_{lw} \): Standard deviation of lateral wander [mm]

\( \sigma_{lwr} \): Reference standard deviation of lateral wander of 200 mm [mm]

\( \sigma_{cs} \): Standardized contact pressure [-]

\( \sigma_c \): Contact pressure of the wheel load [kPa]

\( \sigma_{cr} \): Reference contact pressure of 707 kPa [kPa]

The \( a_p \), \( b_p \) and \( c_p \) values that follow from the individual calculation results are from now on referred to as calculated values. These values, based on the calculations presented in appendix 8.1, form the data-set on which the regression analyses, for the explanation of the values of \( a_p \), \( b_p \) and \( c_p \), are performed. The \( a_p \), \( b_p \) and \( c_p \) values as explained by the regression models that will be discussed in the sections 8.3.2.1 and 8.3.2.2 will be referred to as modelled values.

8.3.2.1 Concrete block pavements with a sand sub-base only.

For the structures with a sand sub-base only the values of \( a_p \) and \( b_p \) have to be described since the value of \( c_p \) equals nil for pavements in which no base layer is present. For describing the value of \( a_p \) and \( b_p \) of block pavements with only a sand sub-base equations 8.3 and 8.4 are applied.
\[ a_p = \left( t_{a_1} + t_{a_2} \times \left( \frac{E_{0r}}{E_0} \right)^{t_{a_3}} \times e^{-\left( \frac{t_{a_4}}{t_{a_5}} \right)^{t_{a_6}}} \right) \times \sigma_{c_{a_1}} \times \left( \frac{1}{\sigma_{c_{a_2}}} \right)^{t_{a_7}} \times L_s^{t_{a_8}} \]  

\[ b_p = \left( t_{b_1} + t_{b_2} \times \left( \frac{E_{0r}}{E_0} \right)^{t_{b_3}} \times e^{-\left( \frac{t_{b_4}}{t_{b_5}} \right)^{t_{b_6}}} \right) \times \sigma_{c_{b_1}} \times \left( \frac{1}{\sigma_{c_{b_2}}} \right)^{t_{b_7}} \times L_s^{t_{b_8}} \]  

where:

- \( t_{a_1}, t_{a_2} \): Model parameters to take into account the effects of \( t_s \) and \( E_0 \) on the value of \( a_p \) [mm]
- \( t_{a_3}, t_{a_4} \): Model parameters to take into account the effects of \( t_s \) and \( E_0 \) on the value of \( a_p \) [-]
- \( c_{a_1} \): Model parameter to take into account the effect of the wheel load contact pressure on the value of \( a_p \) [-]
- \( L_{a_1} \): Model parameter to take into account the magnitude of the wheel load on the value of \( a_p \) [-]
- \( l_{w_{a_1}} \): Model parameter to take into account the standard deviation of lateral wander on the value of \( a_p \) [-]
- \( t_{b_1}, t_{b_2}, t_{b_3}, t_{b_4} \): Model parameters to take into account the effects of \( t_s \) and \( E_0 \) on the \( b_p \)-value [-]
- \( c_{b_1} \): Model parameter to take into account the effect of the wheel load contact pressure on the value of \( b_p \) [-]
- \( L_{b_1} \): Model parameter to take into account the magnitude of the wheel load on the value of \( b_p \) [-]
- \( l_{w_{b_1}} \): Model parameter to take into account the standard deviation of lateral wander on the value of \( b_p \) [-]
- \( E_0 \): Subgrade modulus [MPa]
- \( E_{0r} \): Reference subgrade modulus of 100 MPa [MPa]
- \( L_s \): Standardized wheel load magnitude [-]
- \( \sigma_{c_{a_2}} \): Standardized wheel load contact pressure [-]
- \( \sigma_{l_{w_{a_2}}} \): Standardized standard deviation of lateral wander [-]
- \( t_s \): Thickness of the sub-base [mm]
- \( t_{s_{0f}} \): Reference sub-base thickness of 200 mm [mm]

To give an impression of the analyses that are performed, some results obtained for the behaviour of pavements with a Zaanweg sand sub-base only are discussed hereafter. In figure 8.4 the fit of the \( a_p \), which gives the rut depth after 1000 wheel load repetitions, for describing the development of \( RD \), is presented. The figure shows that the model (equation 8.3) fits the data obtained for the individual calculations quite well.
Fig 8.4  Fit of the \( a_p \) for the absolute rut depth.

A similar plot is given in figure 8.5. In this plot the fit of the \( b_p \), the exponent value in the rutting model, for describing the absolute rut development in pavements with a Zaanweg sand sub-base only, is presented (equation 8.4). Again a good fit between the modelled \( b_p \)-values and the \( b_p \)-values that follow from the individual calculations is achieved.

Fig 8.5  Fit of the \( b_p \) for the absolute rut depth.

For pavements with a sand sub-base only, the \( c_p \)-value equals nil. This implies that the rutting behaviour of such a pavement is completely described by the \( a_p \)-value and \( b_p \)-value. On the basis of these two values the number of wheel load repetitions at which a certain rut depth is accumulated is thus
known.

The number of wheel load repetitions needed for instance for a 15 mm absolute rut depth, the 15 mm absolute rut depth design life, can thus be determined on the basis of the \( a_p \)-values and \( b_p \)-values that follow from the individual calculations, see appendix 8.1, and on the basis of the modelled \( a_p \)-values and \( b_p \)-values.

In figure 8.6 the modelled 15 mm absolute rut depth design life is plotted against the 15 mm absolute rut depth design life that follows from \( a_p \) and \( b_p \) determined on the basis of the results of the individual calculations. As shown by the figure the modelled design life is rather close to the calculated design life. Especially when the design life is shorter than about 1,000,000 wheel load repetitions a good agreement is found.

![Graph showing modelled and calculated 15 mm absolute rut depth design life.](image)

fig 8.6 Modelled and calculated 15 mm absolute rut depth design life.

It is stated here that figure 8.6 itself is not a direct result of any regression analysis, the figure only gives an impression of the accuracy of the combination of the modelled \( a_p \)-values and \( b_p \)-values with regard to the rutting design life.

The figures 8.7 and 8.8 are similar to the figures 8.4 and 8.5 and give the fit of the modelled \( a_p \) and \( b_p \)-values for the development of the relative rut depth. As is shown, the fit obtained for \( a_p \) is again very good, as is the fit for the \( b_p \)-value.

Similar to figure 8.6, figure 8.9 gives insight into the agreement between the modelled 15 mm relative rut depth design life and the 15 mm relative rut depth design life that follows from the \( a_p \) and \( b_p \) values determined for the

Rutting performance model
individual calculation results. As is shown by this figure the design life is explained quite well by the modelled $a_p$-values and $b_p$-values up to a design life of about 400,000 load repetitions. If the design life becomes larger than about 400,000 wheel load repetitions then serious differences between the modelled design life and the calculated design life are shown by figure 8.9.

Fig 8.7  Fit of the $a_p$ for the relative rut depth.

fig 8.8  Fit of the $b_p$ for the relative rut depth.
The tables 8.3 and 8.4 give the results of the regression analyses of the calculated $a_p$-values and $b_p$-values for all the pavements with a sand sub-base only. As indicated by the tables 8.3 and 8.4 three types of sub-base sand are considered.

The Pascalweg sand is a sand that results in excessive rutting. The Zaanweg sand is a sand that results in moderate rutting. The results that were discussed on the previous pages all refer to pavements with a Zaanweg sand sub-base only. The last sand considered is the Weiver sand, which is a sand that results in only limited rutting.

Table 8.3 and 8.4 thus contain all the data that are necessary to determine the rutting behaviour of a concrete block pavement with a sub-base consisting of sand that has the properties of either Pascalweg sand, Zaanweg sand or Weiver sand. On the basis of these tables and the equations 8.3 and 8.4, the rut depth related design life of such pavements can easily be determined.

The question now is what has to be done in the case the sand to be used doesn't have the characteristics of one of the three above mentioned sand types. Although not presented in this dissertation, the author cooperated in a preliminary study which had the objective to determine the possibilities to correlate mechanical characteristics like $c$ and $\phi$, constants to describe the stress dependency of $M_r$ and constants to describe the stress-dependent development of permanent strain to physical parameters like grading, sharpness of the grains etc. From this study (45) it was concluded that it is
indeed possible to develop such correlations. Further research is however needed and already underway to extend and validate these first correlations.

For the time being, the reader is referred to (45) in which regression equations are given that allow to determine to what extent the sand under investigation has characteristics similar to those of the above mentioned three sand types.
### Table 8.3
Regression results for the $\alpha_s$-value in pavements with a sand sub-base only.

<table>
<thead>
<tr>
<th>Sand</th>
<th>Rut depth type</th>
<th>$L_{a1}$ [-]</th>
<th>$c_{pa1}$ [-]</th>
<th>$l_{wa1}$ [-]</th>
<th>$t_{a1}$ [mm]</th>
<th>$t_{a2}$ [mm]</th>
<th>$t_{a3}$ [-]</th>
<th>$t_{a4}$ [-]</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pascalweg</td>
<td>RD_s</td>
<td>1.3787</td>
<td>0.00259</td>
<td>0.3390</td>
<td>15.2938</td>
<td>51.3778</td>
<td>0.8823</td>
<td>0.7328</td>
<td>0.956</td>
</tr>
<tr>
<td></td>
<td>RD_r</td>
<td>1.2737</td>
<td>0.0000</td>
<td>0.6879</td>
<td>11.4960</td>
<td>40.2472</td>
<td>0.8110</td>
<td>0.8167</td>
<td>0.942</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>RD_s</td>
<td>0.8237</td>
<td>0.3290</td>
<td>0.2560</td>
<td>7.8995</td>
<td>10.4280</td>
<td>1.4624</td>
<td>0.6156</td>
<td>0.973</td>
</tr>
<tr>
<td></td>
<td>RD_r</td>
<td>0.6636</td>
<td>0.3464</td>
<td>0.6293</td>
<td>5.3662</td>
<td>6.1260</td>
<td>1.5338</td>
<td>0.7496</td>
<td>0.964</td>
</tr>
<tr>
<td>Weiver</td>
<td>RD_s</td>
<td>0.8141</td>
<td>0.4134</td>
<td>0.1852</td>
<td>3.2112</td>
<td>7.4393</td>
<td>1.3567</td>
<td>0.6244</td>
<td>0.959</td>
</tr>
<tr>
<td></td>
<td>RD_r</td>
<td>0.5952</td>
<td>0.4588</td>
<td>0.6064</td>
<td>2.1312</td>
<td>3.5314</td>
<td>1.3384</td>
<td>0.7424</td>
<td>0.944</td>
</tr>
</tbody>
</table>

### Table 8.4
Regression results for the $b_s$-value in pavements with a sand sub-base only.

<table>
<thead>
<tr>
<th>Sand</th>
<th>Rut depth type</th>
<th>$L_{b1}$ [-]</th>
<th>$c_{pb1}$ [-]</th>
<th>$l_{wb1}$ [-]</th>
<th>$t_{b1}$ [-]</th>
<th>$t_{b2}$ [-]</th>
<th>$t_{b3}$ [-]</th>
<th>$t_{b4}$ [-]</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pascalweg</td>
<td>RD_s</td>
<td>0.7513</td>
<td>0.0000</td>
<td>0.0786</td>
<td>0.2242</td>
<td>0.6996</td>
<td>0.3341</td>
<td>1.1917</td>
<td>0.776</td>
</tr>
<tr>
<td></td>
<td>RD_r</td>
<td>0.7234</td>
<td>0.0000</td>
<td>0.2845</td>
<td>0.1955</td>
<td>0.4666</td>
<td>0.6166</td>
<td>0.6491</td>
<td>0.6507</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>RD_s</td>
<td>1.0385</td>
<td>0.4822</td>
<td>0.0782</td>
<td>0.2149</td>
<td>0.5330</td>
<td>1.1446</td>
<td>1.8985</td>
<td>0.957</td>
</tr>
<tr>
<td></td>
<td>RD_r</td>
<td>1.5513</td>
<td>0.4646</td>
<td>0.1969</td>
<td>0.1958</td>
<td>1.3331</td>
<td>1.4332</td>
<td>1.3409</td>
<td>0.939</td>
</tr>
<tr>
<td>Weiver</td>
<td>RD_s</td>
<td>0.1593</td>
<td>0.2306</td>
<td>0.1409</td>
<td>0.0978</td>
<td>0.1960</td>
<td>0.5231</td>
<td>1.0430</td>
<td>0.920</td>
</tr>
<tr>
<td></td>
<td>RD_r</td>
<td>0.2622</td>
<td>0.4781</td>
<td>0.3824</td>
<td>0.0652</td>
<td>0.2325</td>
<td>0.7478</td>
<td>0.9331</td>
<td>0.867</td>
</tr>
</tbody>
</table>
8.3.2.2 Concrete block pavements with a base layer

For pavement structures with an unbound base layer and a sand sub-base, three parameters are needed to describe the development of rutting: \( a_p \), \( b_p \) and \( c_p \). These parameters are described by the following models.

\[
a_p = \left( t_{a_5} + t_{a_6} \times \left( \frac{t_{sr2}}{t_s + e a_1 \times E_0} \right) \right)^{e a_2} \times e^{\left( \frac{t_b}{t_s \times t_{br1} \times t_{br2}} \right)^{w a_2}} \times \sigma_{cs} \times \sigma_{bws} \times L_s \times L_{a_2} \tag{8.5}
\]

\[
b_p = \left( t_{b_5} + t_{b_6} \times e^{-\left( \frac{t_b}{t_s \times t_{br1} \times t_{br2}} \right)^{w b_2}} \right) \times \sigma_{cs} \times \sigma_{bws} \times L_s \times L_{b_2} \tag{8.6}
\]

\[
c_p = \left( \frac{t_{c_1}}{10,000} + t_{c_2} \times \left( \frac{t_{sr2}}{t_s + e c_1 \times E_0} \right) \right)^{e c_2} \times e^{\left( \frac{t_b}{t_{br1} \times t_{c_3}} \right)^{w c_1}} \times \sigma_{cs} \times \sigma_{bws} \times L_s \times L_{c_1} \tag{8.7}
\]

where:

- \( t_{a_5}, t_{a_6} \): Model parameters to take into account the effects of \( t_s \), \( t_b \), and \( E_0 \) on the value of \( a_p \) [mm]

- \( t_{a_7}, e a_2 \): Model parameters to take into account the effects of \( t_s \), \( t_b \), and \( E_0 \) on the value of \( a_p \)[-]

- \( e a_1 \): Model parameter to take into account the effects of \( E_0 \) on the value of \( a_p \)[-/MPa]

- \( cpa_2 \): Model parameter to take into account the effect of the wheel load contact pressure on the value of \( a_p \)[-]

- \( L_{a_2} \): Model parameter to take into account the magnitude of the wheel load on the value of \( a_p \)[-]

- \( lwa_2 \): Model parameter to take into account the standard deviation of lateral wander on the value of \( a_p \)[-]

- \( t_{b_5}, t_{b_6}, t_{b_7} \): Model parameters to take into account the effects of \( t_b \) on the value of \( b_p \)[-]

- \( cpb_2 \): Model parameter to take into account the effect of the wheel load contact pressure on the value of \( b_p \)[-]

- \( L_{b_2} \): Model parameter to take into account the magnitude of the wheel load.
the wheel load on the value of $b_p$ [-]

$lwb_2$: Model parameter to take into account the standard deviation of lateral wander on the value of $b_p$ [-]

tc$_1$, tc$_2$: Model parameters to take into account the effects of $t_s$, $t_b$ and $E_0$ on the value of $c_p$ [mm]

tc$_3$, ec$_2$: Model parameters to take into account the effects of $t_s$, $t_b$ and $E_0$ on the value of $c_p$ [-]

ec$_1$: Model parameter to take into account the effects of $E_0$ on the value of $c_p$ [-/MPa]

cpc$_1$: Model parameter to take into account the effect of the wheel load contact pressure on the value of $c_p$ [-]

Lc$_1$: Model parameter to take into account the magnitude of the wheel load on the value of $c_p$ [-]

$\text{lwc}_1$: Model parameter to take into account the standard deviation of lateral wander on the value of $c_p$ [-]

$E_0$: Subgrade modulus [MPa]

$L_s$: Standardized wheel load magnitude [-]

$\sigma_{ss}$: Standardized wheel load contact pressure [-]

$\sigma_{lws}$: Standardized standard deviation of lateral wander [-]

t$_s$: Thickness of the sub-base [mm]

t$_{s2}$: 1,000 mm [mm]

t$_b$: Thickness of the base layer [mm]

t$_{br1}$: 200 mm [mm]

t$_{br2}$: 30 mm [mm]

t$_{br3}$: 50 mm [mm]

The equations 8.5 and 8.6, that explain the parameters $a_p$ and $b_p$, are applied for explaining the development of both the absolute and the relative rut depth. The results of the regression analyses based on the calculated data are shown in the tables 8.5 and 8.6.

For the explanation of $c_p$, the regression analysis considers the average $c_p$, value calculated for the absolute rut depth and the relative rut depth. This was done since the value of $c_p$ determines the moment in $N$ at which a pavement with a base layer starts to fail, i.e., starts to accumulate rut depth at an increasing rate. This type of failure has similar effects on both the average rut depth and the absolute rut depth. The results of the regression analysis on the value of $c_p$ are presented in table 8.7.

In figure 8.10 the fit of the parameter $a_p$ for the absolute rut depth is presented for pavements with a Zaanweg sand sub-base combined with a Pascalweg base material base course. As shown by this figure the value of $a_p$ is very well explained by the model.
<table>
<thead>
<tr>
<th>Sand</th>
<th>Base</th>
<th>Rut depth</th>
<th>La₂ [-]</th>
<th>cpa₂ [-]</th>
<th>lwa₂ [-]</th>
<th>ea₁ [-/MPa]</th>
<th>ea₂ [-]</th>
<th>ta₅ [mm]</th>
<th>ta₆ [mm]</th>
<th>ta₇ [-]</th>
<th>r² [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pascalweg</td>
<td>M. Havelaarweg</td>
<td>RD₄</td>
<td>0.5437</td>
<td>0.1428</td>
<td>0.1640</td>
<td>0.3688</td>
<td>0.1255</td>
<td>0.0000</td>
<td>17.1050</td>
<td>1.1271</td>
<td>0.979</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RD₂</td>
<td>0.3251</td>
<td>0.2866</td>
<td>0.5670</td>
<td>3.8393</td>
<td>0.0059</td>
<td>0.0000</td>
<td>13.8811</td>
<td>0.9619</td>
<td>0.961</td>
</tr>
<tr>
<td>Pascalweg</td>
<td></td>
<td>RD₄</td>
<td>0.5905</td>
<td>0.1038</td>
<td>0.2280</td>
<td>0.9312</td>
<td>0.2374</td>
<td>0.0000</td>
<td>21.0306</td>
<td>1.0527</td>
<td>0.992</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RD₂</td>
<td>0.5132</td>
<td>0.0701</td>
<td>0.6365</td>
<td>0.0004</td>
<td>0.1184</td>
<td>0.0000</td>
<td>12.3628</td>
<td>1.2000</td>
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</tr>
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<td>0.4297</td>
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<td>5.6738</td>
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<td>1.0901</td>
<td>0.973</td>
</tr>
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<td>0.2203</td>
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<td>0.6034</td>
<td>3.7533</td>
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<td>0.0306</td>
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<td>1.0863</td>
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</tr>
<tr>
<td>Weiver</td>
<td>M. Havelaarweg</td>
<td>RD₄</td>
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<td>0.8218</td>
<td>0.1263</td>
<td>9.0789</td>
<td>1.7048</td>
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<td>1.6746</td>
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<td></td>
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<td>RD₂</td>
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<tr>
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<td>5.0349</td>
<td>0.7911</td>
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<td>6.5284</td>
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<td></td>
<td></td>
<td>RD₂</td>
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<td>0.1206</td>
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</tr>
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</table>

Table 8.5 Regression results for th: αₜ-value in pavements with a sand sub-base and a base course.
<table>
<thead>
<tr>
<th>Sand Type</th>
<th>Rut depth type</th>
<th>Base Strength (Ld)</th>
<th>qpb₂ (MPa)</th>
<th>lwbd₂ (MPa)</th>
<th>cph₂ (MPa)</th>
<th>th₂ (MPa)</th>
<th>th₇ (MPa)</th>
<th>th₉ (MPa)</th>
<th>r² ([-])</th>
</tr>
</thead>
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<td>0.1189</td>
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<td>0.08036</td>
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<td>0.1939</td>
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<td>0.0000</td>
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<td>0.2137</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.04621</td>
<td>0.11733</td>
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<td>0.0000</td>
<td>0.0000</td>
<td>0.7899</td>
<td>0.03192</td>
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<td>0.0154</td>
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<td>0.0000</td>
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<td>0.1950</td>
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<td>0.0154</td>
<td>0.03473</td>
<td>9.8549</td>
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</tr>
</tbody>
</table>

Table 8.6: Regression results for the $b$ value in pavements with a sand sub-base and a base course.

Rutting performance model
<table>
<thead>
<tr>
<th>Sand</th>
<th>Base</th>
<th>Lc₁ [-]</th>
<th>cpc₁ [-]</th>
<th>lwc₁ [-]</th>
<th>ec₁ [-/MPa]</th>
<th>ec₂ [-]</th>
<th>tc₁ [mm]</th>
<th>tc₂ [mm]</th>
<th>tc₃ [-]</th>
<th>r² [-]</th>
</tr>
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<tbody>
<tr>
<td>Pascalweg</td>
<td>M. Havelaarweg</td>
<td>11.6950</td>
<td>4.1391</td>
<td>0.8675</td>
<td>0.0000</td>
<td>2.7023</td>
<td>6.3582</td>
<td>2.1696</td>
<td>1.8760</td>
<td>0.926</td>
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<td>7.8660</td>
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<td>1.2638</td>
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<td>5.0000</td>
<td>10.1018</td>
<td>0.1340</td>
<td>1.3794</td>
<td>0.839</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>M. Havelaarweg</td>
<td>9.9751</td>
<td>4.0156</td>
<td>0.6998</td>
<td>13.5619</td>
<td>0.5553</td>
<td>5.4263</td>
<td>22.1190</td>
<td>1.3917</td>
<td>0.897</td>
</tr>
<tr>
<td>Pascalweg</td>
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<td>4.2628</td>
<td>0.4502</td>
<td>0.0000</td>
<td>1.0609</td>
<td>1.9712</td>
<td>1.9919</td>
<td>1.6301</td>
<td>0.951</td>
</tr>
<tr>
<td>Weiver</td>
<td>M. Havelaarweg</td>
<td>3.3174</td>
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<td>0.6984</td>
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<td>5.0000</td>
<td>10.9442</td>
<td>0.7270</td>
<td>0.6649</td>
<td>0.910</td>
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<td>4.7536</td>
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<td>0.7929</td>
<td>9.3403</td>
<td>0.5606</td>
<td>1.1423</td>
<td>0.961</td>
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</tbody>
</table>

*Table 8.7* Regression results for the cₚ-value in pavements with a sand sub-base and a base course.
Fig 8.10  Fit of $a_p$ for the absolute rut depth.

The fit of the value of $b_p$ for the development of the absolute rut depth is presented in figure 8.11. As shown by this figure the value of $b_p$ is not explained so well. Figure 8.11 shows that the values of $b_p$ determined for the pavements with a base course are very small when compared to the values determined for pavements without a base course, see figure 8.5. Apparently the variation in the value of the parameter $b_p$ in case of pavements with a base course is only limited and as a result hard to explain. By considering table 8.6 it is shown that the same holds for the other pavements with a base course.

Fig 8.11  Fit of $b_p$ for the absolute rut depth.
The scales of the figures which show the fit of the $a_p$-value and $b_p$-value equal the scales that were applied for the pavements without a base course, see figures 8.4, 8.5, 8.7 and 8.8. By keeping these scales unchanged the figures give an impression of the magnitude of the $a_p$-values and $b_p$-values of pavements with a base course in comparison to pavements with a sand sub-base only. As shown, both the $a_p$-values and especially the $b_p$-values for pavements with a base course are smaller than was the case for pavements with a sand sub-base only.

In figure 8.12 the fit of the $c_p$-value is presented. As mentioned earlier the modelled $c_p$-value is based on the average of the $c_p$-value determined for the absolute rut development and the $c_p$-value determined for the relative rut development. In figure 8.12 the modelled $c_p$-value is plotted against both the calculated $c_p$-value for the absolute rut depth and the calculated $c_p$-value for the relative rut depth.

Figure 8.12 shows two things. First of all it shows that there are indeed only minor differences between the calculated $c_p$-value for the absolute rut depth and the calculated $c_p$-value for the relative rut depth. Secondly the figure shows that the modelled $c_p$-value fits the calculated data quite well. However the difference between the modelled value and the calculated $c_p$ increases with increasing $c_p$-values.

As is shown by table 8.7, the model (equation 8.7) is also capable in describing the $c_p$-values calculated for other combinations of materials.

![Figure 8.12](image_url)

**fig 8.12** Fit of $c_p$

Knowing the $a_p$-value, $b_p$-value and the $c_p$-value the rut development in a concrete block pavement with a base course is known. This implies that the number of load repetitions, for instance needed to come to a 15 mm absolute rut depth, can be determined.
In figure 8.13 the modelled 15 mm absolute rut depth design life is plotted against the calculated 15 mm rut depth design life for those cases that this design life remains smaller than 100,000,000. The figure itself is not a direct result of any regression analysis but gives insight into the combined effects of the modelled $a_p$-values, $b_p$-values and $c_p$-values.

As was the case for concrete block pavements without a base course the fit between the model and the calculation decreases if the design life increases. For the pavements with a Zaanweg sand sub-base and a Pascalweg base the fit becomes poor if the 15 mm absolute rut depth design life exceeds about 4,000,000 wheel load repetitions.

As will be discussed later it is believed that the model is not able to describe the behaviour of concrete block pavements for more than 2,500,000 load repetitions since the permanent strain triaxial tests were performed up to a maximum of 1,000,000 load repetitions.

![Graph](image)

**Fig 8.13** Modelled and calculated 15 mm absolute rut depth design life.

For the development of the relative rut depth the same regression analyses are performed. In figure 8.14 the fit between the modelled $a_p$-values and the calculated $a_p$-values is presented. As shown by this figure and table 8.5 the model explains the value of $a_p$ very well.

Similar to figure 8.11, figure 8.15 gives the fit between the modelled $b_p$ and the calculated $b_p$ for the relative rut depth. As was the case for the development of the absolute rut depth this fit is not so good, see also table 8.6. Again the explanation is that the $b_p$-values as calculated for pavement structures with a base course remain small in all cases.
fig 8.14  Fit of $a_p$, for the relative rut depth.

fig 8.15  Fit of $b_p$, for the relative rut depth.

As explained earlier, no distinction is made by the model between the $c_p$-value for the absolute rut depth and the $c_p$-value for the relative rut depth. The model, equation 8.7, describes the average $c_p$-value.

Since the values of $a_p$, $b_p$, and $c_p$ are known for the relative rut depth, the 15 mm relative rut depth design life can be determined. In figure 8.16 the modelled 15 mm relative rut depth design life is plotted against the calculated 15 mm relative rut depth design life for those cases that this design life remains smaller than 100,000,000. As mentioned earlier again the plot itself
is not a direct result of any regression analysis and gives insight into the effects of the combination of the modelled values of $a_p$, $b_p$ and $c_p$.

As was the case for the 15 mm absolute rut depth design life, figure 8.16 shows that the fit between the modelled design life and the calculated design life decreases if the design life increases. Again the fit becomes poor if the design life exceeds about 4,000,000 load repetitions. Again it is stated here that the model can not be expected to describe the behaviour of concrete block pavements for more than about 2,500,000 load repetitions since the permanent strain triaxial tests were performed up to 1,000,000 load repetitions.

![Graph showing modelled and calculated 15 mm relative rut depth design life](image)

fig 8.16 Modelled and calculated 15 mm relative rut depth design life.

The data presented in the tables 8.5, 8.6 and 8.7 combined with the equations 8.5, 8.6 and 8.7 make it possible to determine the rutting behaviour of concrete block pavements with either a Pascalweg type base or a M. Havelaarweg type base over a sub-base of Pascalweg sand, Zaanweg sand or Weiver sand.

Also now the question can be raised what one has to do if the base material that has to be used has properties that are different from those of the Pascalweg and M. Havelaarweg base material. Again it can be mentioned that reference (45) contains information about correlating mechanical characteristics of base course materials to their physical characteristics. Also these relations are useful to determine the quality of the base material under investigation in relation to the quality of the two base materials mentioned above.
8.4 MODELLED CONCRETE BLOCK PAVEMENT BEHAVIOUR

In the previous section the results of the regression analyses are discussed. It might be clear that the information that is now accessible by the use of the models explaining the \( a_p \), \( b_p \) and \( c_p \)-values contains all the information needed to determine concrete block pavement rutting behaviour and thus for concrete block pavement design. To give insight into the determined concrete block pavement rutting behaviour some figures are given. Figure 8.17 gives the 15 mm relative rut depth design life as determined for concrete block pavements with a Zaanweg sand sub-base only. In this figure the design life is given as a function of the wheel load, the contact pressure and the thickness of the sand sub-base. The effects of lateral wander and the stiffness of the subgrade are not present in this figure (\( E_0 = 60 \text{ MPa}, c_{lw} = 200 \text{ mm} \)).

![Graph showing design life of concrete block pavements](image)

**Figure 8.17** 15 mm relative rut depth design life for concrete block pavements with a Zaanweg sand sub-base only.

As is shown by figure 8.17 the 15 mm relative rut depth design life of the pavements with a Zaanweg sand sub-base only increases if the sub-base thickness increases. If the sub-base however becomes about 1000 mm thick no further design life increase can be obtained by a further increase of the sub-base thickness.

The design life of a concrete block pavement with only a Zaanweg sand sub-base thus shows a limitation. In other words, if the design life of a
concrete block pavement with a 1000 mm thick Zaanweg sand sub-base only
is not long enough, only the application of a base course will extend this
design life: a greater sand sub-base thickness has hardly any effect on the 15
mm relative rut depth design life.

Figure 8.17 also shows that the design life is strongly affected by the
magnitude of the wheel loads to which the pavement is subjected. The
information given in figure 8.17 allows the comparison of the damaging
effects of various wheel loads. Hereto often use is made of the load
equivalency coefficient, \(m_l\). Such a coefficient describes how many load
repetitions \(N_{st}\) of load \(L_{st}\) are needed in order to obtain the same amount
of damage as caused by \(N_i\) repetitions of load \(L_i\). The equation applied
for this purpose is presented in equation 8.8.

Equation 8.9 shows how the value of \(m_l\) can be retrieved on the basis of
the concrete block pavement design life as a function of the wheel load.

\[
N_{st} = N_i \left( \frac{L_i}{L_{st}} \right)^{m_l} \tag{8.8}
\]

\[
m_l = \frac{\ln \left( \frac{D(L_{st} \times 1.01)}{D(L_{st} \times 0.99)} \right)}{\ln \left( \frac{L_{st} \times 0.99}{L_{st} \times 1.01} \right)} \tag{8.9}
\]

where:
- \(m_l\): load equivalency coefficient [-]
- \(L_{st}\): magnitude of the standard wheel load [kN]
- \(L_i\): magnitude of wheel load number "i" [kN]
- \(N_{st}\): equivalent number of standard wheel loads [-]
- \(N_i\): number of wheel loads "L_i" [-]
- \(D(L_{st} \times 1.01)\): design life of the pavement when subjected to wheel loads
  with a magnitude of \(L_{st} \times 1.01\) [-]

As shown by equation 8.9 the value of \(m_l\) is determined by computing
the pavement design life for two slightly different wheel load magnitudes.
Within these computations all other factors effecting the design life are kept
constant, so that \(m_l\) is purely based on load variations.

In figure 8.18 the value of \(m_l\) is given. As shown by figure 8.18, \(m_l\) is
not a constant. The value of \(m_l\) decreases if the wheel load or the contact
pressure increases. A decrease in the sand sub-base thickness also results in a
decrease of \( m_t \). Combining the figures 8.17 and 8.18 this implies that the value of \( m_t \) decreases if the design life of the concrete block pavement with only a Zaanweg sand sub-base decreases. This means that pavements with only a sand sub-base and a relatively short design life are damaged by both heavy loads and light loads. Pavements with only a Zaanweg sand sub-base with a relatively long design life are mainly damaged by the heavy loads, and are hardly damaged by lighter wheel loads.

![Graph showing load equivalency coefficient \( m_t \) based on the 15 mm relative rut depth design life for pavements with only a Zaanweg sand sub-base.]

**Figure 8.18**  Load equivalency coefficient \( m_t \) based on the 15 mm relative rut depth design life for pavements with only a Zaanweg sand sub-base.

Similar to the load equivalency coefficient a contact pressure equivalency coefficient, \( m_c \), can be retrieved from the models discussed earlier. On the basis of this \( m_c \) the effects of a wheel load at a certain contact pressure can be expressed in an equivalent number of wheel loads with another contact pressure, see equation 8.10.

The value of \( m_c \) is determined by means of equation 8.11, which shows a large resemblance with the earlier discussed equation 8.9.

\[
N_{st} = N_i \left( \frac{\sigma_{c,i}}{\sigma_{c,ct}} \right)^{m_c} \tag{8.10}
\]
\[
\ln \left( \frac{\frac{Dl(\sigma_{c,sl} \times 1.01)}{Dl(\sigma_{c,sl} \times 0.99)}}{\frac{\sigma_{c,sl} \times 0.99}{\sigma_{c,sl} \times 1.01}} \right) = \frac{m_c}{\ln \left( \frac{\sigma_{c,sl} \times 0.99}{\sigma_{c,sl} \times 1.01} \right)}
\]

\( (8.11) \)

where:

- \( m_c \): contact pressure equivalency coefficient [-]
- \( \sigma_{c,sl} \): standard contact pressure [kPa]
- \( \sigma_{c,i} \): contact pressure no. "i" [kPa]
- \( N_i \): number of wheel loads with a contact pressure \( \sigma_{c,i} \) [-]
- \( N_{SL} \): equivalent number of wheel loads with a standard contact pressure \( \sigma_{c,sl} \) [-]
- \( Dl(\sigma_{sl} \times 1.01) \): design life of the pavement when subjected to wheel loads with a contact pressure of \( \sigma_{c,sl} \times 1.01 \) [-]

Similar to the determination of \( m_h \), \( m_c \) is determined on the basis of the pavement design life for two slightly different contact pressures. Within these computations all other factors effecting the design life are again kept constant, so that \( m_c \) is purely based on contact pressure variations.

![Graph](image)

**Figure 8.19** Contact pressure equivalency coefficient \( m_c \) based on the 15 mm relative rut depth design life for pavements with only a Zaanweg sand sub-base.

In figure 8.19 the contact pressure equivalency coefficient is presented for pavements with a Zaanweg sand sub-base only (\( E_0 = 60 \) MPa, \( \alpha_{lw} = 200 \))
As is shown by figure 8.19, $m_c$ is not a constant. As was the case for $m_t$, $m_c$ also decreases if the design life of the concrete block pavement decreases.

To show the effects of the implementation of a base layer on the 15 mm relative rut depth design life (allowable number of load repetitions) figure 8.20 is presented. In this figure the 15 mm relative design life is given as a function of the wheel load, the contact pressure and the thickness of the base course. The stiffness of the subgrade, the amount of lateral wander and the total height of the substructure are constant; $E_0 = 60$ MPa, $\sigma_{lw} = 200$ mm and the height of the substructure equals 1000 mm. The sub-base sand again is Zaanweg sand, the base material is the Pascalweg base material.

As shown by figure 8.20 the design life of a concrete block pavement increases if the thickness of the base course increases, which is valid to a thickness of about 400 mm. A further increase of the Pascalweg base layer thickness does not have any significant effect on the concrete block pavement design life.

![Graph showing the relationship between wheel load, contact pressure, and 15 mm relative rut depth design life for concrete block pavements with a Pascalweg base layer on a Zaanweg sand sub-base.](image)

**Fig 8.20** 15 mm relative rut depth design life for concrete block pavements with a Pascalweg base layer on a Zaanweg sand sub-base.

On the basis of the information presented in figure 8.20 the value of the load equivalency coefficient $m_c$ can be retrieved, see equation 8.9. The $m_t$ that is determined for pavements with a Pascalweg base course layer is presented
in figure 8.21. Again the value of $m_i$ is not constant. It is now found that the
value of $m_i$ decreases if the design life of the pavement increases.

*fig 8.21* $m_i$ based on the 15 mm relative rut depth design life for
designs with a Pascalweg base layer on a Zaanweg sand sub-base.

*fig 8.22* $m_i$ based on the 15 mm relative rut depth design life for
designs with a Pascalweg base layer on a Zaanweg sand sub-base.
A similar figure is obtained for the value of the contact pressure equivalency coefficient \( m_c \), determined on the basis of equation 8.11, see figure 8.22. In this figure it is shown that also \( m_c \) is not constant. As is the case for the value of \( m_i \) for pavement structures with a base layer, \( m_c \) decreases if the design life of the concrete block pavement increases.

Since the values found for \( m_c \) and especially \( m_i \) are very high it can be concluded that the design life of a concrete block pavement with an unbound base layer is very much controlled by the heavy wheel loads, especially those heavy wheel with a high contact pressure.

Figure 8.20 shows that the design life of a concrete block pavement with a Pascalweg base course decreases very rapidly with a decreasing base course thickness if this thickness is smaller than 400 mm. This decrease is much stronger than the decrease of the design life with a decrease of the sand sub-base thickness, see figure 8.17. This implies that a concrete block pavement with a base layer that is not thick enough can have a design life that is smaller than the design life of a similar block pavement with only a Zaanweg sand sub-base. On the other hand, concrete block pavements that have a thick enough base layer will show a design life that is much larger than the design life of a pavement with only a sand sub-base.

In order to give insight into the effects of implementing a base course layer figure 8.23 is given. This figure is made for concrete block pavements with a constant substructure height of 1000 mm placed over a 60 MPa subgrade. In the substructure the Zaanweg sub-base sand and the Pascalweg base material were applied. The figure is based on the 15 mm relative rut depth design life at \( d_{J_{ru}} = 200 \text{ mm} \).

In figure 8.23 lines for a constant ratio "design life with base course / design life with sub-base only" are presented for three levels of the wheel load contact pressure. In the case this ratio equals 1, the 15 mm relative rut depth design life of the pavement with only a 1000 mm sand sub-base equals the 15 mm relative rut depth design life of the pavement with a 1000 mm thick substructure in which a base layer with the indicated thickness is present.

Figure 8.23 indicates that when a pavement is subjected to either limited wheel loads or very heavy wheel loads quite thick base layers are needed in order to prolong the design life of such a pavement compared to the pavement life in case no base is applied. If a pavement is for instance loaded by 30 kN wheel loads with a contact pressure of 500 kPa then a Pascalweg base layer of about 260 mm is needed in order to give the pavement with a base the same design life as a similar pavement with only a Zaanweg sand sub-base.
**fig 8.23** Base layer thickness required to obtain a certain design life prolongation.

<table>
<thead>
<tr>
<th>Base layer thickness</th>
<th>Design life</th>
</tr>
</thead>
<tbody>
<tr>
<td>[mm]</td>
<td>% [-]</td>
</tr>
<tr>
<td>177</td>
<td>100.0</td>
</tr>
<tr>
<td>193</td>
<td>109.0</td>
</tr>
<tr>
<td>209</td>
<td>118.1</td>
</tr>
<tr>
<td>225</td>
<td>127.1</td>
</tr>
<tr>
<td>241</td>
<td>136.2</td>
</tr>
</tbody>
</table>

*table 8.8* Relative design life of a block pavement with a base as a function of the base layer thickness for a 60 kN wheel load with a 700 kPa contact pressure.

Figure 8.23 also shows lines for a constant ratio "design life with base course / design life with sub-base only" of 2, 4, 8 and 16. The position of these lines indicates that only a small increase of the base layer thickness will have a strong positive effect on the design life. When it is decided to implement a base layer, and the investments in the minimum required base layer thickness become evident, it is thus favourable to invest in additional base layer thickness. If a pavement is for instance loaded by 60 kN wheel loads with a 700 kPa contact pressure then a base layer with a thickness of 177 mm is needed to come to a pavement with a base that has the same
design life as the pavement without a base layer. By increasing the thickness of the base by only 16 mm to 193 mm the design life doubles. Adding another 16 mm to the base layer again doubles the design life, see table 8.8.

8.5 PAVEMENT BEHAVIOUR UNDER REAL TRAFFIC

In the previous sections it is shown that the behaviour of a concrete block pavement depends on the substructure design, the stiffness of the subgrade and the wheel loads that are applied to the pavement. The values of \( m_i \) and \( m_c \) vary with the magnitude of the standard wheel load \( "L_{st}" \) and the standard contact pressure "\( \sigma_{c,st} \)" at which they are determined. Furthermore the values of \( a_p \), \( b_p \) and \( c_p \) vary with the magnitude of the wheel load and the contact pressure.

A real pavement is loaded by a spectrum of wheel loads. Since the behaviour of a concrete block pavement partly depends on the wheel loads that are applied to the pavement, the design of pavements loaded by such a spectrum of wheel loads becomes a somewhat complicated matter. In this section it is discussed how the effects of various wheel loads with various contact pressures on the behaviour of a concrete block pavement are taken into account.

The basic equations that completely describe the behaviour of a concrete block pavement under a spectrum of wheel loads are.

\[
RD_p = a_p \left( \frac{N_{st}}{1000} \right)^{b_p} + c_p \left( e^{d_p \left( \frac{N_{st}}{1000} \right)} - 1 \right) \tag{8.12}
\]

\[
N_{eq} = \sum_{i=1}^{i_{max}} \frac{\%_i}{100} \times \left( \frac{L_i}{L_{st}} \right)^{m_1} \times \left( \frac{\sigma_{c,i}}{\sigma_{c,st}} \right)^{m_2} \tag{8.13}
\]

\[
N_{st} = \sum_{i=1}^{i_{max}} N \times \frac{\%_i}{100} \times \left( \frac{L_i}{L_{st}} \right)^{m_1} \times \left( \frac{\sigma_{c,i}}{\sigma_{c,st}} \right)^{m_2} \quad \text{or} \quad N_{st} = N \times N_{eq} \tag{8.14}
\]

\[
RD_a = RP \times RD_p \tag{8.15}
\]

where:
- \( L_{st} \): standard wheel load [kN]
- \( \sigma_{c,st} \): standard contact pressure [kPa]
i_{max}:
number of different wheel loads in the wheel load spectrum trafficking the pavement [-]  
\%
percentage of wheel load "i" in the spectrum [-]  
L_i:
magnitude of wheel load number "i" in the spectrum [kN]  
σ_{c,i}:
contact pressure of wheel load number "i" in the spectrum [kPa]  
m_i:
wheel load equivalency coefficient [-]  
m_c:
contact pressure equivalency coefficient [-]  
N_{eq}:
average number of equivalent standard wheel loads per load repetition according to equation 8.13 [-]  
N_{eq}:
equivalent standard wheel load repetitions [-]  
N:
number of wheel load repetitions according to the actual wheel load spectrum [-]  
RD_a:
absolute rut depth [mm]  
RD_r:
relative rut depth or rut depth underneath a 1.2 m straight edge [mm]  
RP:
RD_a/RD_r - ratio [-]  
a_p, c_p:
 rutting model parameters [mm]  
b_p, d_p:
rutting model parameters [-]  

In the previous section it was explained that the behaviour of a concrete block pavement loaded by a single type of wheel load with magnitude "L_i," and a contact pressure "σ_{c,i}" can be determined on the basis of regression models that explain the values of the parameters a_p, b_p and c_p (d_p = 0.2 x c_p) for both the absolute rut depth and the relative rut depth. This implies that the design life of a concrete block pavement loaded by a single type of wheel load is known as a function of the relative rut depth at failure, RD_{r,f}. This design life under a single type of load "dl_i(L_i,σ_{c,i},RD_{r,f})" equals the design life presented in the figures 8.17 and 8.20. Based on this knowledge the parameters needed for the complete description of the rutting behaviour of a pavement loaded by a wheel load spectrum, see equations 8.12 to 8.15, can be retrieved. The most important equation in this process directly gives the design life of a pavement trafficked by a spectrum of wheel loads.

\[
dl_i(L_i,σ_{c,i},RD_{r,f}) = \frac{1}{\sum_{i=1}^{i_{max}}\left(\frac{\%_i}{100}\right)}
\]

(8.16)

where:
\(dl_i(L_i,σ_{c,i},RD_{r,f})\):
design life of the pavement for an allowable relative rut depth "RD_{r,f}" when trafficked by the wheel load spectrum under consideration [-]  

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design life of the pavement for an allowable relative rut depth \( \text{RD}_{r,t} \) when trafficked by the wheel load \( "i" \) with a magnitude \( L_i \) and a contact pressure \( \sigma_{c,i} \) [-].

**RD\(_{r,t}\)**: allowable relative rut depth or allowable rut depth underneath a 1.2 m straight edge [mm]

On the basis of equation 8.16 it becomes relatively easy to determine the load equivalency coefficient, \( m_i \). Hereto first the design life of the pavement is determined for the case that all wheel loads in the spectrum are multiplied by 1.01, \( d_{l_i}(L_i \times 1.01, \sigma_{c,i}, \text{RD}_{r,t}) \).

\[
d_{l_i}(L_i \times 1.01, \sigma_{c,i}, \text{RD}_{r,t}) = \frac{100}{\sum_{i=1}^{\text{max}} \left( \frac{\%_i}{d_{l_i}(L_i \times 1.01, \sigma_{c,i}, \text{RD}_{r,t})} \right)}
\]  

(8.17)

Similar to this also the design life of the pavement for the situation that all wheel loads are multiplied by 0.99 can be determined. On the basis of both \( d_{l_i}(L_i \times 1.01, \sigma_{c,i}, \text{RD}_{r,t}) \) and \( d_{l_i}(L_i \times 0.99, \sigma_{c,i}, \text{RD}_{r,t}) \) the value of the load equivalency coefficient, \( m_i \), under the total spectrum of wheel loads is hereafter determined with an equation that closely resembles equation 8.9.

\[
m_i = \frac{\ln \left( \frac{d_{l_i}(L_i \times 1.01, \sigma_{c,i}, \text{RD}_{r,t})}{d_{l_i}(L_i \times 0.99, \sigma_{c,i}, \text{RD}_{r,t})} \right)}{\ln \left( \frac{0.99}{1.01} \right)}
\]  

(8.18)

where:

- \( d_{l_i}(L_i \times 1.01, \sigma_{c,i}, \text{RD}_{r,t}) \): design life of the pavement loaded by the wheel load spectrum under consideration in which all wheel loads are multiplied by 1.01 [-]

- \( d_{l_i}(L_i \times 0.99, \sigma_{c,i}, \text{RD}_{r,t}) \): design life of the pavement loaded by the wheel load spectrum under consideration in which all wheel loads are multiplied by 0.99 [-]

Similar to the load equivalency coefficient, the contact pressure coefficient, \( m_c \), can be determined, see equation 8.19.
\[
\ln \left( \frac{dl_i(L_t, \sigma_{c,t} \times 1.01, RD_{r,f})}{dl_i(L_t, \sigma_{c,t} \times 0.99, RD_{r,f})} \right) \\
\ln \left( \frac{0.99}{1.01} \right)
\]

\[m_c = \frac{\ln \left( \frac{dl_i(L_t, \sigma_{c,t} \times 1.01, RD_{r,f})}{dl_i(L_t, \sigma_{c,t} \times 0.99, RD_{r,f})} \right)}{\ln \left( \frac{0.99}{1.01} \right)} (8.19)\]

where:
\[dl_i(L_t, \sigma_{c,t} \times 1.01, RD_{r,f}): \text{ design life of the pavement loaded by the wheel load spectrum under consideration in which all contact pressures are multiplied by 1.01 [-]}\]
\[dl_i(L_t, \sigma_{c,t} \times 0.99, RD_{r,f}): \text{ design life of the pavement loaded by the wheel load spectrum under consideration in which all contact pressures are multiplied by 0.99 [-]}\]

As becomes clear by considering the equations 8.12 to 8.14 the values of both \(L_{st}\) and \(\sigma_{c, st}\) are really of no importance. The values of \(a_p\) and \(d_p\) however depend on the values of \(L_{st}\) and \(\sigma_{c, st}\). It is therefore only logical to chose a \(L_{st}\) and \(\sigma_{c, st}\) that are representative for the wheel load spectrum that traffics the pavement. Such a representative value depends on both the wheel load spectrum trafficking the pavement and the pavement's reaction to it in terms of rutting. For the standard wheel load magnitude, \(L_{st}\), such a representative value is determined as follows.

\[L_{st} = \sum_{i=1}^{i_{max}} \left( L_i \times \frac{\%_i}{100} \times \frac{dl_i(L_t, \sigma_{c,t}, RD_{r,f})}{dl_i(L_t, \sigma_{c,t}, RD_{r,f})} \right) (8.20)\]

Similar to \(L_{st}\), a representative value of the standard contact pressure, \(\sigma_{c, st}\), is determined as follows.

\[\sigma_{c, st} = \sum_{i=1}^{i_{max}} \left( \sigma_{c,t} \times \frac{\%_i}{100} \times \frac{dl_i(L_t, \sigma_{c,t}, RD_{r,f})}{dl_i(L_t, \sigma_{c,t}, RD_{r,f})} \right) (8.21)\]

On the basis of \(L_{st}\), \(\sigma_{c, st}\), \(m_i\) and \(m_c\) the equivalent number of standard wheel loads introduced by a single fictitious wheel load in which the total wheel load spectrum is taken into account, \(N_{eq}\), can be determined by means of equation 8.13.

In order to determine the values of \(a_p\), \(b_p\), \(c_p\) and \(d_p\) four sets of numbers of standard wheel load repetitions and corresponding rut depths are needed. Hereafter these four sets are defined.

\[RD_1 = 0.25 \times RD_{r,f}, \quad RD_2 = 0.5 \times RD_{r,f}\]
\[RD_3 = 1 \times RD_{r,f}, \quad RD_4 = 1.5 \times RD_{r,f}\] (8.22)
\[ N_{st1} = N_{eq} \times \frac{dl(L_p \sigma_{c,p} RD_1)}{1000} \]
\[ N_{st2} = N_{eq} \times \frac{dl(L_p \sigma_{c,p} RD_2)}{1000} \]
\[ N_{st3} = N_{eq} \times \frac{dl(L_p \sigma_{c,p} RD_3)}{1000} \]
\[ N_{st4} = N_{eq} \times \frac{dl(L_p \sigma_{c,p} RD_4)}{1000} \]

(8.23)

where:
- \( N_{st1}, N_{st2}, N_{st3}, N_{st4} \): number of standard wheel load repetitions divided by 1000 at which a relative rut depth of \( RD_1, RD_2, RD_3 \) and \( RD_4 \) respectively developed [-]
- \( RD_1, RD_2, RD_3, RD_4 \): four levels of relative rut depth taken into consideration [mm]

In the case that the value of \( c_p \) and \( d_p \) will equal nil (pavement with a sand sub-base only) the values of \( a_p \) and \( b_p \) are determined quite easy as follows.

\[ b_p = \frac{\ln \left( \frac{RD_3}{RD_4} \right)}{\ln \left( \frac{N_{st3}}{N_{st4}} \right)} \]  
(8.24)

\[ a_p = \frac{RD_4}{N_{st4} b_p} \]  
(8.25)

In the case that the values of \( c_p \) and \( d_p \) will not per definition equal nil (pavement with a base layer) the situation becomes slightly more complex. In these cases an iterative process is needed in order to determine \( a_p, b_p, c_p \) and \( d_p \).

The iterative process starts with \( d_p = 0 \) and a small \( c_p \) value of for instance 0.000001 mm. The first step is to determine the value of \( b_p \), equation 8.26. Hereafter the value of \( a_p \) is determined by means of equation 8.27.

Hereafter the value of \( d_p \) can be determined on the basis of the calculated \( a_p \)-value and \( b_p \)-value in combination with the \( c_p \)-value that follows from the previous step in the iteration process, see equation 8.28. The last step in the process is to calculate the value of \( c_p \) on the basis of the calculated values of \( a_p, b_p \) and \( d_p \), see equation 8.29.

After the value of \( d_p \) is determined the process is repeated. This time the actual calculated \( c_p \) and \( d_p \) values are used to determine the values of both \( a_p \),

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and $b_p$. The process is repeated until the $c_p$ and $d_p$ values that follow from the process equal those with which the process was started.

$$b_p = \left( \frac{\ln \left( \frac{RD_1 - c_p \times (\exp(d_p \times N_{st1}) - 1)}{RD_2 - c_p \times (\exp(d_p \times N_{st2}) - 1)} \right)}{\ln \left( \frac{N_{st1}}{N_{st2}} \right)} \right)^{b_p}$$  \hspace{1cm} (8.26)

$$a_p = \frac{RD_2 - c_p \times (\exp(d_p \times N_{st2}) - 1)}{N_{st2}^{b_p}}$$  \hspace{1cm} (8.27)

$$d_p = \left( \frac{\ln \left( \frac{RD_3 - a_p \times N_{st3}^{b_p}}{RD_4 - a_p \times N_{st4}^{b_p}} \right)}{N_{st3}^{b_p} - N_{st4}^{b_p}} \right)^{d_p}$$  \hspace{1cm} (8.28)

$$c_p = \frac{RD_4 - a_p \times N_{st4}^{b_p}}{\exp(d_p \times N_{st4}) - 1}$$  \hspace{1cm} (8.29)

All parameters needed to describe the relative rutting behaviour of a pavement loaded by a spectrum of wheel loads are now known. If the development of the absolute rut depth is also of importance the value of the parameter $RP$ has to be determined too, this last parameter is determined using the following equation.

$$RP = \sum_{i=1}^{i_{max}} \frac{\%_i}{100} \times \frac{dl_i(L_i, \sigma_i, RD_{r_i})}{dl_i(L_i, \sigma_i, RD_{r_i})} \times \frac{RD_{a_i}(dl_i(L_i, \sigma_i, RD_{r_i}))}{RD_{r_i}}$$  \hspace{1cm} (8.30)

where:

$RD_{a_i}(dl_i(L_i, \sigma_i, RD_{r_i}))$: absolute rut depth introduced by $dl_i(L_i, \sigma_i, RD_{r_i})$ wheel load repetitions of wheel load "i" [mm]

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8.6 DESIGN EXAMPLES

In the previous sections the development of equations that describe the rutting behaviour of concrete block pavements with and without an unbound base has been discussed. The value of the various parameters in these equations are determined for three sand types and two types of base material. On the basis of the rutting behaviour of a concrete block pavement of course the rut depth related design life can easily be determined so that the equations discussed enable the rut depth related design of concrete block pavements.

Although it is possible to use a pocket computer to solve the necessary equations, it is of course much more appropriate to use a P.C. for this. For that reason a simple Turbo Pascal program is given in appendix 8.2 which is helpful for the potential user in solving his particular design problem. The program determines the nine parameters that completely describe a pavement’s rutting behaviour \((a_p, b_p, c_p, d_p, L_{st}, m_l, \sigma_{c,st}, \bar{m}_{c}\) and RP, see equations 8.12 to 8.15) as a function of the subgrade stiffness, the substructure design and the wheel load spectrum.

If a concrete block pavement is only designed on the basis of the number of load repetitions until a certain rut depth has been obtained, then these 9 parameters contain much more information than needed. In these cases only equation 8.16 has to be applied, which directly gives the design life of a concrete block pavement trafficked by a spectrum of wheel loads.

The mentioned 9 parameters are only needed if more information than strictly the rut depth related design life is required. If one wants to know the effects of delayed maintenance, for instance, the development of rutting beyond the design life becomes of interest. The rutting parameters are also needed for the determination of traffic induced longitudinal unevenness, see chapter 9.

In this section some design examples are discussed. The axle load distribution used in the examples is taken from literature (14), see table 8.9. This axle load spectrum does not give any information about the contact pressure of the various axle loads. In the last column of table 8.9 estimated contact pressures are however given. These contact pressures are calculated assuming that the contact area of any wheel load is a circular surface with a radius of 150 mm.

In the following design examples the purpose is to design a concrete block pavement that has a design life of 5 years. It is assumed that 150,000 axle load repetitions with the distribution given in table 8.9 are applied to the pavement per year per lane. It is assumed that the stiffness of the subgrade equals 60 MPa and that traffic only shows minor lateral wander, \(\sigma_{\text{w}} = 200\) mm.
<table>
<thead>
<tr>
<th>axle load group [kN]</th>
<th>average axle load [kN]</th>
<th>average wheel load [kN]</th>
<th>wheel load spectrum % [-]</th>
<th>contact pressure (150 mm) [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 - 20</td>
<td>10</td>
<td>5</td>
<td>60.80</td>
<td>70.7</td>
</tr>
<tr>
<td>20 - 40</td>
<td>30</td>
<td>15</td>
<td>12.75</td>
<td>212.2</td>
</tr>
<tr>
<td>40 - 60</td>
<td>50</td>
<td>25</td>
<td>15.75</td>
<td>353.7</td>
</tr>
<tr>
<td>60 - 80</td>
<td>70</td>
<td>35</td>
<td>6.40</td>
<td>495.1</td>
</tr>
<tr>
<td>80 - 100</td>
<td>90</td>
<td>45</td>
<td>2.75</td>
<td>636.6</td>
</tr>
<tr>
<td>100 - 120</td>
<td>110</td>
<td>55</td>
<td>1.20</td>
<td>778.1</td>
</tr>
<tr>
<td>120 - 140</td>
<td>130</td>
<td>65</td>
<td>0.30</td>
<td>919.6</td>
</tr>
<tr>
<td>140 - 160</td>
<td>150</td>
<td>75</td>
<td>0.04</td>
<td>1061.0</td>
</tr>
<tr>
<td>&gt; 160</td>
<td>170</td>
<td>85</td>
<td>0.01</td>
<td>1202.5</td>
</tr>
</tbody>
</table>

*Table 8.9* The axle load or wheel load distribution.

First a design is made for a pavement that only has a Zaanweg sand sub-base. The design criterion is a 15 mm relative rut depth. The design comes down to the determination of the required sub-base thickness. Hereto the behaviour of the pavement is determined for various sub-base thicknesses, see *table 8.10*.

<table>
<thead>
<tr>
<th>( t_s ) [mm]</th>
<th>( a_p ) [mm]</th>
<th>( b_p ) [-]</th>
<th>( L_{st} ) [kN]</th>
<th>( m_t ) [-]</th>
<th>( \sigma_{cst} ) [kPa]</th>
<th>( m_c ) [-]</th>
<th>RP [-]</th>
<th>dl [-]</th>
<th>dl [year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>6.012</td>
<td>0.528</td>
<td>53.99</td>
<td>3.913</td>
<td>764</td>
<td>1.466</td>
<td>1.35</td>
<td>128,130</td>
<td>0.854</td>
</tr>
<tr>
<td>600</td>
<td>6.208</td>
<td>0.436</td>
<td>57.19</td>
<td>4.659</td>
<td>809</td>
<td>1.751</td>
<td>1.39</td>
<td>259,530</td>
<td>1.730</td>
</tr>
<tr>
<td>700</td>
<td>6.329</td>
<td>0.378</td>
<td>59.62</td>
<td>5.329</td>
<td>843</td>
<td>2.006</td>
<td>1.44</td>
<td>470,711</td>
<td>3.138</td>
</tr>
<tr>
<td>800</td>
<td>6.378</td>
<td>0.345</td>
<td>61.29</td>
<td>5.815</td>
<td>867</td>
<td>2.191</td>
<td>1.48</td>
<td>725,174</td>
<td>4.834</td>
</tr>
<tr>
<td>900</td>
<td>6.381</td>
<td>0.329</td>
<td>62.31</td>
<td>6.120</td>
<td>882</td>
<td>2.305</td>
<td>1.51</td>
<td>955,763</td>
<td>6.372</td>
</tr>
<tr>
<td>1000</td>
<td>6.361</td>
<td>0.321</td>
<td>62.89</td>
<td>6.294</td>
<td>890</td>
<td>2.370</td>
<td>1.53</td>
<td>1,124,287</td>
<td>7.495</td>
</tr>
<tr>
<td>1100</td>
<td>6.333</td>
<td>0.317</td>
<td>63.20</td>
<td>6.390</td>
<td>894</td>
<td>2.404</td>
<td>1.54</td>
<td>1,232,725</td>
<td>8.218</td>
</tr>
</tbody>
</table>

*Table 8.10* Rutting behaviour as a function of the sub-base thickness.

In the first column "\( t_s \)" of *table 8.10* the Zaanweg sand sub-base thickness is given. The 7 columns hereafter give the parameters that completely describe the pavements rutting behaviour (\( c_p = 0 \) and \( d_p = 0 \), no base layer is present). In the column "dl [-]" the 15 mm relative rut depth design life, expressed in the number of wheel load repetitions, is given. The last column gives the design life expressed in years.
As shown by table 8.10 the 15 mm relative design life increases with increasing sub-base thickness. The required design life of 5 years is obtained by applying a 900 mm sub-base (design life 6.4 years). The table furthermore shows that the value RP increases with an increase of the sub-base thickness. This implies that the shape of the rut bowls that will develop becomes smoother if the sub-base thickness increases, see figure 8.24.

![Diagram](image)

**fig 8.24** Relation between RP and the rut shape.

To show the effects of the use of a base layer on the pavement rutting behaviour under the wheel load spectrum, some additional design calculations are made. These additional calculations all consider the Pascalweg base layer over a Zaanweg sand sub-base. The results of the calculations are presented in table 8.11 and they show the enormous positive effect of a base layer on the 15 mm relative rut depth design life.

As shown by the first column in table 8.11 the pavements with a Pascalweg base layer all have a 900 mm thick substructure, which equals the thickness of the Zaanweg sand sub-base needed to assure the required design life when no base layer is applied. In figure 8.25 the positive effects of implementing a base are shown in a plot that gives the ratio of the design life of the pavements with a 900 mm substructure in which a Pascalweg base is implemented over the design life of the pavement with only a 900 mm Zaanweg sand sub-base.
### Table 8.11 Rutting behaviour of the pavement when a base layer is applied.

<table>
<thead>
<tr>
<th>Sub-base thickness [mm]</th>
<th>$a_c$ [mm]</th>
<th>$c_e$ [mm]</th>
<th>$L_{st}$ [kN]</th>
<th>$\sigma_{cst}$ [kPa]</th>
<th>RP [-]</th>
<th>dl [-]</th>
<th>dl [years]</th>
</tr>
</thead>
<tbody>
<tr>
<td>575</td>
<td>4.732</td>
<td>18.849</td>
<td>83.55</td>
<td>1182</td>
<td>1.149</td>
<td></td>
<td>558,679</td>
</tr>
<tr>
<td>275</td>
<td>0.048</td>
<td>4.3388</td>
<td>23.466</td>
<td>7.621</td>
<td></td>
<td></td>
<td>3.725</td>
</tr>
<tr>
<td>550</td>
<td>4.491</td>
<td>7.230</td>
<td>82.95</td>
<td>1174</td>
<td>1.210</td>
<td></td>
<td>2,791,040</td>
</tr>
<tr>
<td>300</td>
<td>0.051</td>
<td>1.4301</td>
<td>21.658</td>
<td>7.047</td>
<td></td>
<td></td>
<td>18.607</td>
</tr>
<tr>
<td>525</td>
<td>4.243</td>
<td>3.077</td>
<td>82.38</td>
<td>1165</td>
<td>1.259</td>
<td></td>
<td>9,905,380</td>
</tr>
<tr>
<td>325</td>
<td>0.056</td>
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**Figure 8.25** Design life prolongation as a result of the use of a base layer.

As indicated by figure 8.25 a Pascalweg base layer of about 280 mm thick is needed in order to obtain a pavement that has a design life that equals the design life of a pavement with a 900 mm Zaanweg sand sub-base only. With a further increase of the base layer thickness a very strong increase of the design life is observed. By the application of a 300 mm thick base layer
on a 550 mm Zaanweg sand sub-base, the design life of the pavement increases from 955,763 wheel load repetitions to 2,791,040 wheel load repetitions or from 6.37 years to 18.61 years respectively.

The design calculations thus show that when it is decided to implement a base layer, this layer should have a certain minimum thickness in order to realize the required increase of the pavement design life. In this example, using a combination of Zaanweg sand and Pascalweg base material for pavements trafficked by the wheel load spectrum given in table 8.9, the minimum base layer thickness is about 280 mm. By only investing in an additional 20 mm of base the design life of the pavement can almost be tripled.

Finally figure 8.26 presents the development of both the relative rut depth, i.e. the rut depth underneath a 1.2 m straight edge, and the absolute rut depth in the two pavements with a 900 mm thick substructure.

![Graph showing rut development](image)

**Fig 8.26** Rut development in two pavements due to the wheel load spectrum given in table 8.9.

### 8.7 RANGES OF VALIDITY

In the previous sections a rutting performance model was discussed. This model will give the rutting behaviour of concrete block pavements with a sand sub-base only and pavements with an unbound base layer and a sand sub-base. In developing the model three types of sands and two types of base materials were considered.

The model gives the rutting behaviour as a function of the sub-base
thickness and sub-base sand, the base layer thickness and base layer material, the subgrade modulus, the standard deviation of lateral wander and the wheel load spectrum applied to the pavement. Regardless of the various input-values the model will always give the rutting behaviour of the pavement in question.

The accuracy of the explained rutting behaviour however depends on the input-values. The rutting performance model is based on calculations in which the permanent strain development of the various materials in the substructure plays an important part. The permanent strain development applied in the calculations is based on laboratory measurements in which 1,000,000 load repetitions are applied to a sample.

This implies that the explained rutting behaviour of a concrete block pavement after more than 1,000,000 wheel load repetitions is based on extrapolation of laboratory data. It is believed that a minor extrapolation of 6.7% will hardly effect the accuracy of the model. Considering a log-scale for the load repetitions applied in the laboratory, this implies that the rutting performance model is accurate up to 2,500,000 wheel load repetitions (log(N) 6 -> 6.4).

The calculations in which the development of permanent strain in the substructure is explained, are based on a resilient finite element analysis of a concrete block pavement. This implies that the theory and thus the model is only valid in those cases that the behaviour of a concrete block pavement can be considered as being resilient. In other words the accumulated rut depth per wheel load repetition should be limited. Concrete block pavements that have a very short, unrealistic design life will certainly not show resilient behaviour. For such pavements the rutting performance model may thus not be applicable.

The calculations performed to come to the rutting performance model all consider concrete block pavements with 80 mm thick concrete blocks. The stiffness of the joints is based on the concrete block layer behaviour as measured for rectangular blocks with a length of 211 mm and a width of 105 mm in herringbone bond.

The model, as a result, explains the rutting behaviour of concrete block pavements constructed with rectangular blocks, with the earlier mentioned dimensions, in herringbone bond.

As stated earlier, it was assumed that when a base layer is present a 50 mm thick crusher sand bedding layer is projected between the base and the concrete blocks. In the case of the application of a base layer the rutting performance model thus gives the rutting behaviour assuming that the bedding sand layer has a 50 mm thickness.
8.8 CONCLUSIONS

The rutting performance model discussed in this chapter gives a complete description of the rut development in concrete block pavements with a sand sub-base only and concrete block pavements with a sand sub-base and an unbound base layer. The model takes into account the effects of the substructure design, the subgrade modulus, the wheel load spectrum applied to the pavement and the amount of lateral wander.

The required model parameters are determined for pavements with three types of sub-base sand and two types of base layer materials. The effects of the quality of the granular materials used in the substructure on the rut development can thus be examined on the basis of the rutting performance model.

Since the model does not only give the rut depth related design life, but the complete rutting behaviour of the concrete block pavement under consideration the outcome of a design calculation can be used for the further analysis of the block pavement behaviour under dynamic wheel loads, see chapter 9.

A powerful tool for the design of concrete block pavements with a substructure in which only granular materials are present has thus been developed.

On the basis of the model some practical conclusions can be drawn. First of all it can be concluded that the behaviour of a pavement is not only determined by the substructure design and the subgrade modulus, but also by the wheel load spectrum that is applied to the pavement. As a result a residential street can not be designed on the basis of the pavement behaviour under heavy commercial traffic.

Secondly it is concluded that thin base layers should not be implemented in concrete block pavements since such base layers might have a negative effect on the design life, compared to the design life of pavements with a sand sub-base only. The minimum base layer thickness required to obtain a prolongation of the design life depends on the wheel loads applied to the pavement and the contact pressure of these wheel loads, see figure 8.23. Furthermore it is concluded that when a base layer is implemented, only minor additional investments in the base layer thickness will have a strong positive effect on the pavements design life.

Thirdly it is found that the amount of lateral wander effects the behaviour of a concrete block pavement. Since the amount of lateral wander is related to the width of the pavement this implies that an optimal pavement width will exist which is related to construction and maintenance costs.
Fourthly it is found that the design life of a pavement with a Zaanweg sand sub-base only increases with the thickness of the sub-base. It is however found that the application of sub-bases thicker than 1000 mm no longer results in a prolongation of the design life.

For the base layer thickness a similar conclusion can be drawn. Again the design life of a pavement increases with the thickness of the applied base layer. For the Pascalweg base material it is observed that the application of a base thicker than about 400 mm will no longer result in an increase of the design life.

Fifthly it is found that the load equivalency coefficient, $m_l$, and the contact pressure equivalency coefficient, $m_c$, vary with both the pavement design and the wheel load spectrum applied to the pavement.

For concrete block pavements with a Zaanweg sand sub-base only it was found that both $m_l$ and $m_c$ decrease with a decrease of the sub-base thickness and thus of the design life. This implies that the development of rutting in concrete block pavements with a thick sand sub-base only is mainly determined by the large wheel loads in the wheel load spectrum, especially when these wheel loads have a high contact pressure. For concrete block pavements with a thin Zaanweg sand sub-base the values of both $m_l$ and $m_c$ are much smaller, implying that these pavements are also damaged by the smaller wheel loads applied by relatively low contact pressures.

For pavements with a Pascalweg base in combination with a Zaanweg sub-base the situation is completely different. These pavements show an increase of $m_l$ and $m_c$ with a decrease of the base layer thickness. This implies that thin base layers are very sensitive for overloading. Large wheel loads now result in a rapid introduction of rut depth, especially when these wheel loads have a high contact pressure.

With an increase of the base layer thickness $m_l$ and $m_c$ show a decrease. This implies that thicker base layers are not so easily overloaded. Even for concrete block pavements with thicker Pascalweg base layers the values of $m_l$ and $m_c$ however remain large ($m_l$ = about 15, $m_c$ = about 5), which implies that rutting in pavement structures with a Pascalweg base layer is mainly determined by large wheel loads with high contact pressures.
Appendix 8.1

Individual rut depth calculations

In this appendix the input for the individual rut depth calculations made for the development of the rutting performance model is given as a list of figures. Each set of figures refers to one calculation and contains 8 figures. These figures refer to the following: calculation number [-], total substructure thickness [mm], base layer thickness [mm], subgrade modulus [MPa], wheel load magnitude [kN], radius of the wheel load contact area [mm], contact pressure [kPa] and standard deviation of lateral wander [mm].

The first set of 99 individual rut depth calculations refers to pavements with a sand sub-base only. On the bases of the rut development computed in these 99 calculations the values of $a_p$ and $b_p$ are determined for both the development of $R_{D_a}$ and $R_D$ by means of regression analyses. The values of $c_p$ and $d_p$ for rutting in pavements with only a sand sub-base equal nil. The calculations for pavements with only a sand sub-base are made for sub-bases made of Pascalweg sand, Zaanweg sand and Weiver sand.

After the first set of 99 individual rut depth calculations a second set of 201 individual rut depth calculations is presented. This second set refers to the calculations that consider pavements with a base and a sub-base. In this second set of calculations a 50 mm crusher sand bedding layer was projected between the concrete blocks and the base layer. The second set of calculations was performed 6 times, for the three sand types mentioned above in combination with both a Pascalweg base and a M. Havelaarweg base. On the basis of the results of the individual calculations performed for pavements with a base layer the values of $a_p$, $b_p$, $c_p$ and $d_p$ are determined for both the development of $R_{D_a}$ and $R_D$ by means of regression analyses.

Individual calculations considering pavements with a sand sub-base only.

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264 Appendix 8.1: Individual rut depth calculations
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Appendix 8.1: Individual rut depth calculations

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<tr>
<td>198</td>
<td>1500</td>
<td>750</td>
<td>60</td>
<td>50</td>
<td>150.0</td>
<td>707.4</td>
<td>300</td>
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<tr>
<td>199</td>
<td>1500</td>
<td>750</td>
<td>30</td>
<td>50</td>
<td>150.0</td>
<td>707.4</td>
<td>100</td>
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<td>200</td>
<td>1500</td>
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<td>50</td>
<td>150.0</td>
<td>707.4</td>
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<td>201</td>
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<td>150.0</td>
<td>707.4</td>
<td>300</td>
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</table>

**Appendix 8.1: Individual rut depth calculations**
Appendix 8.2
Program listing

In this appendix the listing of a simple Turbo Pascal program is given. The program computes the rutting parameters $a_p$, $b_p$, $c_p$, $d_p$, $L_{sn}$, $q_{sn}$, $m_1$, $m_c$ and RP as a function of the substructure design, the subgrade modulus, the wheel load spectrum applied to the pavement, and the amount of lateral wander.

```pascal
program rutting_performance_model;
uses dos, crt;
var i, sand, base: integer;
   ok: boolean;
lspec, sspec, spec: array[1..20] of real;
nspec: array[1..20] of extended;
ch: char;
e0, ts, th, slw, rdf: real;
la12, cpa12, lwa12, ta15, ta26, ta37, ta4, lb12, cpb12, lwb12, tb15, tb26, tb37, tb5: real;
n1, n2, ea1, ea2, lc1, cpe1, lwc1, ec1, ec2, tc1, tc2, tc3: real;
rde, rp, ap, bp, cp, dp, neq, lst, cst, ml, mc, a, b, c, d: extended;

function power(a, b: extended): extended;
var  pow: extended;
begin
  if a > 0 then pow := exp(b * ln(a))
  else if a = 0 then begin
    pow := 0;
  end;
power := pow;
end;

procedure wait;
begin
  writeln; writeln('PRESS ANY KEY TO PROCEED!!!!');
  repeat until keypressed;
  ch := readkey;
  if ch = #0 then ch := readkey;
end;

procedure readsspectrum;
var sspec: real;
begin
```

Appendix 8.2: Program listing 267
clrscr;
writeln;
writeln('In the following screen you must give the wheel load spectrum to which the');
writeln('pavement that is to be designed will be subjected. The maximum number of');
writeln('different wheel loads is 20. By giving in a wheel load of 0 kN you will end');
writeln('the input procedure.);
wait;
repeat
  i:=1;
  ok:=false;
clrscr;
write('no [-]:  Wheel load [kN]:  Contact pressure [kPa]:  Percentage [-]:');
while ok=false do begin
  gotoxy(1,1+i);write(i:2);
  gotoxy(11,1+i);
  read(lspec[i]);
  gotoxy(11,1+i);
  write(lspec[i]:6:2,','');
  if lspec[i]>0 then begin
    gotoxy(30,1+i);
    read(sspec[i]);
    gotoxy(30,1+i);
    write(sspec[i]:6:1,','');
    gotoxy(57,1+i);
    read(spec[i]);
    gotoxy(57,1+i);
    writeln(spec[i]:10:2,'');
    i:=i+1;
    if i=21 then ok:=true;
  end else ok:=true;
end;
i:=i;
while i<=20 do begin
  sspec[i]:=0;lspec[i]:=0;spec[i]:=0;
  i:=i+1;
end;
sumspec:=0;
for i:=1 to 20 do sumspec:=sumspec+spec[i];
for i:=1 to 20 do spec[i]:=100*spec[i]/sumspec;
i:=1;
while (lspec[i]>0) and (i<=20) do begin
  gotoxy(57,5+i);
  writeln(spec[i]:5:2,'');
  i:=i+1;
end;
gotoxy(1,1+i);
writeln('');
write('Do you agree with this wheel load spectrum (Y/N):');
ch:=readkey;
if (ch='y') or (ch='Y') then ok:=true else begin
  if ch='n' then ch:=readkey;
  ok:=false;
end;
until ok=true;
procedure materials;
begin
  clrscr;
  writeln('What type of sand do you want to use for the sub-base? Give "1" for a good');
  writeln('type of sand (Weiver sand), "2" for an average type of sand (Zaanweg sand);');
  write('or "3" for a poor sand (Pascalweg sand): ');
  ok:=false;
  while not ok do begin
    gotoxy(42,3);read(sand);
    if (sand = 3) and (sand >= 1) then ok:=true else begin
      gotoxy(42,3);write('');
    end;
  end;
  writeln;
  writeln('What type of base material do you want to use for the base layer if any? Give');
  writeln('type of base material (Pascalweg base material) or "2" for a poor base');
  write('material (M. Havehaarweg base material): ');
  ok:=false;
  while not ok do begin
    gotoxy(35,8);read(base);
    if (base = 2) or (base = 1) then ok:=true else begin
      gotoxy(35,8);write('');
    end;
  end;
end;

procedure readin(r:char);
var sandbase:integer;
begin
  sandbase:=10*sand;
  if tb > 0 then sandbase:=sandbase+base;
  case sandbase of
  10:begin
    if ri = 'a' then begin
      la12:=0.8141;cpa12:=0.4134;lw12:=0.1852;ta15:=3.2112;ta26:=7.4393;ta37:=1.3567;
      ta4:=0.6244;tb12:=0.1593;cpb12:=0.2306;lw12:=0.1409;tb15:=0.0978;tb26:=0.1960;
      tb37:=0.5231;tb5:=1.0430;
    end else if ri = 't' then begin
      la12:=0.5952;cpa12:=0.4558;lw12:=0.6064;ta15:=2.1312;ta26:=3.5314;ta37:=1.3384;
      ta4:=0.7424;tb12:=0.2622;cpb12:=0.4781;lw12:=0.3824;tb15:=0.0652;tb26:=0.2625;
      tb37:=0.7478;tb5:=0.9331;
    end;
  20:begin
    if ri = 'a' then begin
      la12:=0.8237;cpa12:=0.3290;lw12:=0.2560;ta15:=7.8995;ta26:=10.4280;ta37:=1.4624;
      ta4:=0.6156;tb12:=1.0385;cpb12:=0.4822;lw12:=0.0782;tb15:=0.2149;tb26:=0.5330;
      tb37:=1.1446;tb5:=1.8985;
    end;
end;
end else if ri = 'r' then begin

la12: = 0.6636; cpal2: = 0.3464; lwa12: = 0.6293; ta15: = 5.3662; ta26: = 6.1260; ta37: = 1.5338;
ta4: = 0.7496; lb12: = 1.5513; cpbl2: = 0.4646; lwb12: = 0.1969; tb15: = 0.1958; tb26: = 1.3331;
tb37: = 1.4332; tb5: = 1.3409;
end;
end;
30:begin
if ri = 'a' then begin

la12: = 1.3787; cpal2: = 0.00259; lwa12: = 0.3390; ta15: = 15.2938; ta26: = 51.3778;
ta37: = 0.8823; ta4: = 0.7328; lb12: = 0.7513; cpbl2: = 0.0000; lwb12: = 0.0786; tb15: = 0.2242;
tb26: = 0.6996; tb37: = 0.3341; tb5: = 1.1917;
end else if ri = 'r' then begin

la12: = 1.2737; cpal2: = 0.0000; lwa12: = 0.6879; ta15: = 11.4960; ta26: = 40.2472;
ta37: = 0.8110; ta4: = 0.8167; lb12: = 0.7234; cpbl2: = 0.0000; lwb12: = 0.2845; tb15: = 0.1955;
tb26: = 0.4666; tb37: = 0.6166; tb5: = 0.6491;
end;
end;
11:begin
if ri = 'a' then begin

la12: = 0.52830; cpal2: = 0.1531; lwa12: = 0.1495; ea1: = 5.0349; ea2: = 0.7911;
ta15: = 0.3467; ta26: = 6.52837; ta37: = 0.9117; lb12: = 0.41297; cpbl2: = 0.2447; lwb12: = 0.0000;
tb15: = 0.09667; tb26: = 0.01992; tb37: = 5.0632; lc1: = 7.6574; cpcl1: = 4.7536; lwc1: = 0.3960;
ec1: = 0.00000; ec2: = 0.7929; tc1: = 9.3403; tc2: = 0.5606; tc3: = 1.1423;
end else if ri = 'r' then begin

la12: = 0.2931; cpal2: = 0.2457; lwa12: = 0.6295; ea1: = 2.7290; ea2: = 0.4711; ta15: = 0.1206;
ta26: = 3.4867; ta37: = 0.9386; lb12: = 0.9023; cpbl2: = 0.6126; lwb12: = 0.7803; tb15: = 0.04862;
tb26: = 0.22789; tb37: = 2.9359; lc1: = 7.6574; cpcl1: = 4.7536; lwc1: = 0.3960; ec1: = 0.0000;
ec2: = 0.7929; tc1: = 9.3403; tc2: = 0.5606; tc3: = 1.1423;
end;
end;
12:begin
if ri = 'a' then begin

la12: = 0.1402; cpal2: = 0.2818; lwa12: = 0.1263; ea1: = 9.0789; ea2: = 1.7048; ta15: = 0.4213;
ta26: = 2.1957; ta37: = 1.67457; lb12: = 0.0806; cpbl2: = 0.1970; lwb12: = 0.0000; tb15: = 0.09091;
tb26: = 0.04541; tb37: = 2.4411; lc1: = 3.3174; cpcl1: = 7.8969; lwc1: = 0.6984; ec1: = 16.2835;
ec2: = 5.0000; tc1: = 10.9442; tc2: = 0.7270; tc3: = 0.6649;
end else if ri = 'r' then begin

la12: = 0.0000; cpal2: = 0.06565; lwa12: = 0.60237; ea1: = 7.0885; ea2: = 0.9496; ta15: = 0.1491;
ta26: = 0.8538; ta37: = 1.8977; lb12: = 0.1008; cpbl2: = 0.6146; lwb12: = 0.8666; tb15: = 0.04018;
tb26: = 0.00000; tb37: = 5.3102; lc1: = 3.3174; cpcl1: = 7.8969; lwc1: = 0.6984; ec1: = 16.2835;
ec2: = 5.0000; tc1: = 10.9442; tc2: = 0.7270; tc3: = 0.6649;
end;
end;
21:begin
if ri = 'a' then begin

la12: = 0.3847; cpal2: = 0.2310; lwa12: = 0.1901; ea1: = 6.3864; ea2: = 0.4511; ta15: = 0.2822;
ta26: = 10.8289; ta37: = 1.1981; lb12: = 0.1950; cpbl2: = 0.3988; lwb12: = 0.0000; tb15: = 0.08136;
tb26: = 0.08233; tb37: = 9.1550; lc1: = 13.0615; cpcl1: = 4.2628; lwc1: = 0.4502; ec1: = 0;
ec2: = 1.0609; tc1: = 1.9712; tc2: = 1.9919; tc3: = 1.63014;
end else if ri = 'r' then begin

la12: = 0.1509; cpal2: = 0.2874; lwa12: = 0.6193; ea1: = 3.7671; ea2: = 0.3508; ta15: = 0.0306;
ta26: = 7.4058; ta37: = 1.0863; lb12: = 0.0000; cpbl2: = 0.8140; lwb12: = 0.2698; tb15: = 0.03294;
tb26: = 0.10780; tb37: = 9.8549; lc1: = 13.0615; cpcl1: = 4.2628; lwc1: = 0.4502; ec1: = 0;
ec2: = 1.0609; tc1: = 1.9712; tc2: = 1.9919; tc3: = 1.63014;
end;
end;

22:begin
if ri = 'a' then begin
  la12 := 0.4297;cpa12 := 0.2894;lw12 := 0.1684;ea1 := 5.6738;ea2 := 0.3556;ta15 := 0.0000;
  ta26 := 11.5001;ta37 := 1.0901;lb12 := 0.3507;cpb12 := 0.2457;lw12 := 0.0000;tb15 := 0.08098;
  tb26 := 0.040787;tb37 := 8.5684;lc1 := 9.9751;cpcl := 4.0156;lwcl := 0.6998;cc1 := 13.5619;
  ec2 := 0.5553;tc1 := 5.4263;tc2 := 22.1190;tc3 := 1.3917;
end else if ri = 'r' then begin
  la12 := 0.22029;cpa12 := 0.3754;lw12 := 0.6034;ea1 := 3.7533;ea2 := 0.1734;ta15 := 0.0000;
  ta26 := 7.74397;ta37 := 0.94925;lb12 := 0.7899;cpb12 := 0.0154;lw12 := 0.6982;
  tb15 := 0.03192;tb26 := 0.03473;tb37 := 7.7915;lc1 := 9.9751;cpcl := 4.0156;lwcl := 0.6998;
  cc1 := 13.5619;ec2 := 0.5553;tc1 := 5.4263;tc2 := 22.1190;tc3 := 1.3917;
end;
end;

31:begin
if ri = 'a' then begin
  la12 := 0.5905;cpa12 := 0.1038;lw12 := 0.2280;ea1 := 0.9312;ea2 := 0.2374;ta15 := 0.0000;
  ta26 := 21.0306;ta37 := 1.0527;lb12 := 0.0000;cpb12 := 0.1714;lw12 := 0.0000;tb15 := 0.08558;
  tb26 := 0.09453;tb37 := 11.3170;lc1 := 7.8660;cpcl := 3.6005;lwcl := 1.2638;cc1 := 0.00000;
  ec2 := 5.0000;tc1 := 10.1018;tc2 := 0.13399;tc3 := 1.3794;
end else if ri = 'r' then begin
  la12 := 0.5132;cpa12 := 0.0701;lw12 := 0.6365;ea1 := 0.0004;ea2 := 0.1184;ta15 := 0.0000;
  ta26 := 12.3628;ta37 := 1.2000;lb12 := 0.0000;cpb12 := 0.1939;lw12 := 0.2317;tb15 := 0.04621;
  tb26 := 0.11733;tb37 := 11.4337;lc1 := 7.8660;cpcl := 3.6005;lwcl := 1.2638;cc1 := 0.00000;
  ec2 := 5.0000;tc1 := 10.1018;tc2 := 0.13399;tc3 := 1.3794;
end;
end;

32:begin
if ri = 'a' then begin
  la12 := 0.5437;cpa12 := 0.1428;lw12 := 0.1640;ea1 := 0.3688;ea2 := 0.1255;ta15 := 0.0000;
  ta26 := 17.1050;ta37 := 1.1271;lb12 := 0.0000;cpb12 := 0.1189;lw12 := 0.0000;tb15 := 0.08036;
  tb26 := 0.09910;tb37 := 11.0121;lc1 := 11.6950;cpcl := 4.1391;lwcl := 0.8675;cc1 := 0.00000;
  ec2 := 2.7023;tc1 := 6.3582;tc2 := 2.1696;tc3 := 1.8760;
end else if ri = 'r' then begin
  la12 := 0.3251;cpa12 := 0.2866;lw12 := 0.5670;ea1 := 3.8393;ea2 := 0.0059;ta15 := 0.0000;
  ta26 := 13.8811;ta37 := 0.9619;lb12 := 0.0000;cpb12 := 0.1593;lw12 := 0.2564;tb15 := 0.03717;
  tb26 := 0.09189;tb37 := 11.5312;lc1 := 11.6950;cpcl := 4.1391;lwcl := 0.8675;cc1 := 0.00000;
  ec2 := 2.7023;tc1 := 6.3582;tc2 := 2.1696;tc3 := 1.8760;
end;
end;
end;

procedure detabcd(Load, lw, cont, ts, tb, Emod: extended);
var emacht, ied, dh, ls, scs, lws: real;
begin
ls := load/50;
scs := cont/707.355302;
lws := lw/200;
if tb = 0 then begin
  a := ta15 + ta26*power(100/Emod, ta37)*exp(-power(ts/(200*ta4), ta4));
end;
end;

Appendix 8.2: Program listing 271
a:=a*power(lsl,la12)*power(scs,cpa12)*power(1/lwa12);
b:=tb15+tb26*power(100/ecomd,tc37)*exp(-power(ts/(200*tb5),tb5));
b:=b*power(lsb,lb12)*power(scs,cpb12)*power(1/lwb12);
c:=0;
d:=c*200/1000000;
end else begin
  dh:=ls*ta37*200;
  ied:=power(1000/(ts+ecl*ecomd),ca2);
  emacht:=power((tb/dh),ta37);
  if emacht > 10000 then emacht:=10000;
a:=ta15+ied*ta26*exp(-emacht);
a:=a*power(lsl,la12)*power(scs,cpa12)*power(1/lwa12);
dh:=ls*ta37*30;
  emacht:=power((tb/dh),tb37);
  if emacht > 10000 then emacht:=10000;
b:=tb15+tb26*exp(-emacht);
b:=b*power(lsb,lb12)*power(scs,cpb12)*power(1/lwb12);
dh:=50*tc3;
  ied:=power(1000/(ts+ecl*ecomd),cc2);
  emacht:=power((tb/dh),tc3);
  if emacht > 10000 then emacht:=10000;
c:=(ct1/10000)+ied*tc2*exp(-emacht);
c:=c*power(lsc,lc1)*power(scs,cpe1)*power(1/lwc1);
d:=c*200/1000000;
end;
end;

function Nlife(Load,lw,cont,ts,tb,E0,rdepth:extended):extended;
var fh,n1,n2,sd,d1,d2,lwm:extended;

begin
dctabcd(Load,lw,cont,ts,tb,E0);
if (tb=0) or (c=0) then begin
  nlife:=1000*power(rdepth/a,1/b);
end else begin
  n1:=0.1;
  sd:=a*power(n1/1000,b)+c*(exp(d*n1)-1);
  while sd < rdepth do begin
    n2:=10*n1;
    sd:=a*power(n2/1000,b)+c*(exp(d*n2)-1);
    n1:=n2;
  end;
  n1:=n1/10;
  while (n2-n1) > 1e-15*n2 do begin
    sd:=a*power(n1+n2)/2000,b)+c*(exp(d*(n1+n2)/2)-1);
    if sd > rdepth then n2:=(n1+n2)/2 else n1:=(n1+n2)/2;
  end;
  nlife:=n1;
end;
end;

procedure detnspec(f1,fc,rdepth:real);

Appendix 8.2: Program listing
begin
  i := 1;
  while (spec[i] > 0) and (i <= 20) do begin
    nspec[i] := Nlife(ll*spec[i], slw, fc*sspec[i], ts, tb, E0, rdepth);
    i := i + 1;
  end;
  while i < 20 do begin
    nspec[i] := 0;
    i := i + 1;
  end;
end;

function detdesignlife: real;
var ngr: real;
begin
  ngr := 0;
  i := 1;
  while (spec[i] > 0) and (i <= 20) do begin
    ngr := ngr + spec[i]/nspec[i];
    i := i + 1;
  end;
  detdesignlife := 100/ngr;
end;

procedure detapbpcdp;
var dc1, dc2, av, bv, cv, dv, rdc1, rdc2, rdc3, rdc4, rd1, rd2, rd3, rd4, nn1, nn2, nn3, nn4: real;
begin
  rd1 := rdf/4;
  rd2 := rdf/2;
  rd3 := rdf;
  rd4 := rdf*1.5;
  write(rd1:10:5,' ', rd2:10:5,' ', rd3:10:5,' ', rd4:10:5);
  writeln(rd4:10:5);
  detspec(1, 1, rd1); nn1 := detdesignlife/neq/1000;
  detspec(1, 1, rd2); nn2 := detdesignlife/neq/1000;
  detspec(1, 1, rd3); nn3 := detdesignlife/neq/1000;
  detspec(1, 1, rd4); nn4 := detdesignlife/neq/1000;
  if c=0 then begin
    bp := ln(rd3/rd4)/ln(nn3/nn4);
    cp := 0; dp := 0;
    ap := rdf/power(nn3, bp);
    rdc1 := ap*power(nn1, bp) + cp*(exp(nn1*dp)-1);
    rdc2 := ap*power(nn2, bp) + cp*(exp(nn2*dp)-1);
    rdc3 := ap*power(nn3, bp) + cp*(exp(nn3*dp)-1);
    rdc4 := ap*power(nn4, bp) + cp*(exp(nn4*dp)-1);
    write(rdc1:10:5, ' ', rdc2:10:5, ' ', rdc3:10:5, ' ');
    writeln(rdc4:10:5);
  end else begin
    dp := 0; cp := 0.00001;
    av := 0; bv := 0; cv := cp; dv := dp;
end;

Appendix 8.2: Program listing  273
ok := false;
while not ok do begin
    bp := ln((rd1-cp*(exp(dp*nn1)-1))/(rd2-cp*(exp(dp*nn2)-1)));
    bp := bp/ln(nn1/nn2);
    if bp < 0 then bp := 0;
    ap := (rd2-cp*(exp(dp*nn2)-1))/power(nn2,bp);
    dp := ln((rd3-ap*power(nn3,bp))/(rd4-ap*power(nn4,bp)))/(nn3-nn4);
    cp := (rd3-ap*power(nn3,bp))/(exp(dp*nn3)-1);
    dc1 := 0*dp; dc2 := 10*dp;
    rdc4 := ap*power(nn4,bp) + cp*(exp(nn4*dp)-1);
    while abs(rdc4-rd4)/rd4 > 1e-8 do begin
        dp := (dc1 + dc2)/2;
        cp := (rd3-ap*power(nn3,bp))/(exp(dp*nn3)-1);
        rdc4 := ap*power(nn4,bp) + cp*(exp(nn4*dp)-1);
        if rdc4 > rd4 then dc2 := dp else dc1 := dp;
    end;
    if (abs(ap-apv) < 0.0000000001) and
        (abs(bp-bpv) < 0.0000000001) and
        (abs(cp-cpv) < 0.0000000001) and
        (abs(dp-dpv) < 0.0000000001) then ok := true;
    av := ap; bv := bp; cv := cp; dv := dp;
    end;
end;

function detr(nf, rdf:real):real;
var rdin:extended;
    rde, rph:real;
begin
    rph := 0;
    for i := 1 to 20 do begin
        if spc[i] = 0 then begin
            if nspec[i] < 40000000 then begin
                readin('a');
                databc[dspec[i], slw, sspec[i], ts, tb, E0];
                rdin := a*power(nspec[i]/1000,b) + c*(exp(d*nspec[i])-1);
                rph := rph + (spec[i]/100)*(n1/(spec[i])*(rdin/rdf));
            end else begin
                readin('r');
                databc[dspec[i], slw, sspec[i], ts, tb, E0];
                rde := a*power(4000000/1000,b) + c*(exp(d*4000000)-1);
                readin('a');
                databc[dspec[i], slw, sspec[i], ts, tb, E0];
                rdin := a*power(4000000/1000,b) + c*(exp(d*4000000)-1);
            end;
        end;
    end;
end;
rph: = rph + (spec[i]/100)*(n1/nspec[i])*(rdin/rdend);
    end;
    end;
    end;
    detrp: = rph;
    readin('r');
end;

procedure design:

begin
    clrscri;
    writeln;
    writeln('In the following screen you will be able to give in the subgrade modulus, the');
    writeln('thickness of the sand sub-base, the thickness of the base if any, the amount');
    writeln('of lateral wander and the relative rut depth design criterion. The program');
    writeln('will hereafter give the rutting properties of the pavement when loaded by the');
    writeln('wheel load spectrum you gave in.');
    ok: = false;
    while not ok do begin
        clrscri;
        writeln('Give the sub-grade modulus [MPa]: ');
        readin(E0);
        writeln('Give the sand sub-base thickness [mm]: ');
        readin(ts);
        writeln('Give the base layer thickness [mm]: ');
        readin(tb);
        writeln('Give the standard deviation of lateral wander [mm]: ');
        readin(slw);
        writeln('Give the relative rut depth design criterion [mm]: ');
        readin(rdf);
        readin('r');
        detnspec(1.01, 1.0, rdf);
        n1: = detdesignlife;
        detnspec(0.99, 1.0, rdf);
        n2: = detdesignlife;
        ml: = ln(n2/n1)/ln(1.01/0.99);
        detnspec(1.01, 1.0, rdf);
        n1: = detdesignlife;
        detnspec(1.0, 0.99, rdf);
        n2: = detdesignlife;
        mc: = ln(n2/n1)/ln(1.01/0.99);
        detnspec(1, 1.0, rdf);
        n1: = detdesignlife;
        lst: = 0; cst: = 0; neq: = 0;
        i: = 1;
        while (i <= 20) do begin
            if spec[i] > 0 then begin
                lst: = lst + (spec[i]/100)*n1*ispec[i]/nspec[i];
                cst: = cst + (spec[i]/100)*n1*sspec[i]/nspec[i];
            end;
            i: = i + 1;
        end;
end;
neq:=0;
i:=1;
while (i <= 20) do begin
  if spec[i] > 0 then begin
    neq:=neq+(spec[i]/100)*power(lspec[i]/lst,ml)*power(sspec[i]/cst,mc);
  end;
i:=i+1;
end;
detupbcpdp;
detspec(1,1,rdf);
n1:=detdesignlift;
rp:=detrp(n1,rdf);
c1scr;
writeln;
writeln;'RUTTING BEHAVIOUR OF THE PAVEMENT UNDER THE GIVEN WHEEL LOAD SPECTRUM:';
writeln;
writeln;'Standard wheel load, Lst:
  lst:7;2, 'kN');
Write(("Standard contact pressure, ocst:     
      'cst:7;2, 'atm');
writep('Load equivalency coefficient, ml:  
      'ml:7;3, '-');
writeln('Contact pressure equivalency coefficient, mc:  
      'mc:7;3, '-');
writeln('a-value:  
      'ap:6;3, 'mm');
writeln('b-value:  
      'bp:6;4, '-');
writeln('c-value:  
      'cp:7;4, 'mm');
writeln('d-value:  
      'dp:8;6, '-');
writeln('RDa/RDr-ratio, rp:  
      'rp:6;4, '-');
writeln;
writeln;'Nequivalent per load repetition, Neq:  
      'neq:7;4, '-');
writeln;'Design life in standard load repetitions:  
      'n1*:neq:10;0, '-');
writeln;'Design life in spectrum load repetitions:  
      'n1:10;0, '-');
rde:=ap*power(n1*neq/1000,bp)+cp*(exp(n1*neq*dp/1000)-1);
writeln('Relative rut depth at end of design life:  
      'rde:5;2, 'mm');
writeln('Absolute rut depth at end of design life:  
      'rde*rp:5;2, 'mm');
writeln;
writeln;'Do you want to determine the behaviour of a concrete block pavement with?';
writeln('another substructure design (y/n) ? ');
ch:=readkey;
if (ch = 'y') or (ch = 'Y') then ok:=false else begin
  if ch = '#0 then ch:=readkey;
  ok:=true;
end;
begin
  readspectrum;
  materials;
  design;
end.
9

Prediction of longitudinal unevenness in concrete block pavements

9.1 INTRODUCTION

All pavements have a certain degree of longitudinal unevenness or roughness, even directly after construction. This means that vehicles that travel over any pavement will introduce dynamic axle loads that vary over the length of the pavement. Since the rut depth that is introduced by a vehicle not only depends on the properties of the pavement but also on the axle load to which it is subjected, a vehicle will always introduce a rut depth that is varying over the length.

The rut depth variations resulting from repeated vehicle passages can have a strong effect on the roughness of a pavement. In certain circumstances the roughness that follows from rut depth variations can lead to premature pavement failure.

To get insight into this particular type of pavement damage, repeated vehicle simulations are performed. Within these simulations the dynamic loads caused by every individual vehicle that is travelling over a pavement are calculated in order to obtain the accumulated effects of traffic on longitudinal unevenness.

In this chapter the development of a model describing the effects of repeated vehicle passages on the development of traffic induced longitudinal unevenness is also discussed. This model has a few special features which allow a strong reduction of the number of vehicle simulations needed. As a result the model is much faster than the repeated vehicle simulation and is considered to be very valuable for use in practice.
9.2 VEHICLE SIMULATION

9.2.1 The model

To be able to quantify the effects of dynamic axle loads on the development of longitudinal unevenness in concrete block pavements the dynamic axle loads, that develop when a vehicle is travelling over a pavement, have to be known.

Longitudinal road profiles are mostly measured and considered as two dimensional profiles. The profile height is measured over the length of the pavement and sometimes no distinction is made between the profile height in the left and in the right wheeltrack. It is however clear that the right wheels most likely travel over a different longitudinal profile then the left wheels.

The longitudinal profiles that were measured for this research are two dimensional too and do not distinguish the differences between profile height in the left and right wheeltrack. As mostly is the case for such longitudinal profiles, the measured longitudinal profiles represent the average of the longitudinal profiles in both the wheeltracks, see figure 9.1.

![Diagram](image)

**fig 9.1** A single longitudinal profile of a road mostly represents the average longitudinal profile.

Since the longitudinal profiles considered in this research do not distinguish the difference between the profile heights in the left and right wheeltrack a half vehicle model is needed for the simulation of a vehicle. The model is presented in figure 9.2.

As shown by figure 9.2 the model of the half vehicle really is a three-
mass-spring system in which the mass of the front axle including the tires is represented by the mass \( M_{tf} \) [kg]. The mass \( M_{tr} \) [kg] represents the mass of the rear axle including tires.

The tires themselves deform when they are loaded. In this model the tires have a linear stiffness, represented by springs with stiffnesses \( k_{sf} \) [N/m] and \( k_{sr} \) [N/m] for the front and rear tires respectively.

The suspension system between the carriage work of the vehicle and the axles consists of a shock absorber which is represented by the linear dampers \( C_{sf} \) [Ns/m] and \( C_{sr} \) [Ns/m] and a suspension spring represented by the linear springs \( k_{sf} \) [N/m] and \( k_{sr} \) [N/m].

The carriage work of the vehicle including the payload, if any, is represented by the third mass \( M_{cw} \) [kg]. This mass connects the front and rear axles and thus has a length that equals the distance between the centre of the front and the rear wheels. As a result of the dimensions of this mass it will have a certain moment of inertia \( J_{cw} \) [kg m²]. The centre of gravity of this mass is located \( l_f \) [m] behind the front axle and \( l_r \) [m] in front of the rear axle. Of course the vehicle must have a horizontal speed \( v \) [m/s] in order to develop dynamic effects.

If the vehicle is travelling over an uneven surface dynamic effects will occur. These effects will cause the three masses to move in the vertical direction with speeds of \( v_{tf}(t), v_{tr}(t), v_{cw}(t) \) [m/s] for the front tire, the rear tire and the centre of gravity of the carriage work respectively. The associated accelerations with which the masses change their vertical speed are \( a_{tf}(t), a_{tr}(t), a_{cw}(t) \) [m/s²] again for the front tire, the rear tire and the centre of gravity of the carriage work respectively. The pitch angle of the carriage

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work is referred to as \( \phi_{cw}(t) \) [-] so that the pitch rotational speed becomes \( \phi'_{cw}(t) \) [-/s] and the change in pitch rotational speed becomes \( \phi''_{cw}(t) \) [-/s^2].

The simulation of the vehicle means that the values \( v_{t}(t) \), \( v_{n}(t) \), \( v_{cw}(t) \), \( a_{t}(t) \), \( a_{n}(t) \), \( a_{cw}(t) \), \( \phi'_{cw}(t) \) and \( \phi''_{cw}(t) \) are to be determined for each point on the longitudinal profile. Hereto a number of equations are derived. Within these equations use is only made of the most simple rules of dynamics given in the equations 9.1 and 9.2.

\[
\frac{dv}{dt} = a = \frac{F}{M} \quad \text{and} \quad \frac{da}{dt} = \frac{dF/dt}{M} \quad (9.1)
\]

\[
\frac{d\phi}{dt} = \phi' = \frac{F \times \text{arm}}{J} \quad \text{and} \quad \frac{d\phi'}{dt} = \frac{dF/dt \times \text{arm}}{J} \quad (9.2)
\]

where:

- \( v \): speed [m/s]
- \( a \): acceleration [m/s^2]
- \( F \): force [N]
- \( M \): mass [kg]
- \( \phi' \): rotational speed [-/s]
- \( \phi'' \): rotational acceleration [-/s^2]
- \( \text{arm} \): length of the arm over which a force works [m]
- \( J \): moment of inertia [kg m^2]
- \( t \): time [s]

On the basis of the rules of dynamics given in equations 9.1 and 9.2 matrix "[A]" can be derived, see equation 9.3. [A] gives the relation between values of: \( v_{t}(t) \), \( v_{n}(t) \), \( v_{cw}(t) \), \( a_{t}(t) \), \( a_{n}(t) \), \( a_{cw}(t) \), \( \phi'_{cw}(t) \) and \( \phi''_{cw}(t) \) and their changes in time, see equation 9.4.

In equation 9.4, also the vertical speeds are present at which the bottom sides of the springs representing the tires are forced to move. These values "\( v_{yr}(t) \) [m/s]" and "\( v_{yr}(t) \) [m/s]" for the movements of the front and rear tire respectively are the input that cause the three mass spring system to react to the longitudinal profile.
\[ [A] = \begin{bmatrix}
0 & 1 & 0 & 0 & 0 & 0 & 0 \\
-k_{sr} & -c_{sr} & -k_{sr} & c_{sr} & 0 & 0 & 0 \\
\frac{k_{sr}}{M_{cw}} & \frac{c_{sr}}{m_{cw}} & \frac{k_{sr}}{M_{tr}} & \frac{c_{sr}}{m_{tr}} & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & 0 \\
\frac{k_{sf}}{M_{tr}} & \frac{c_{sf}}{M_{tr}} & \frac{-k_{sf} - k_{tr}}{M_{tr}} & \frac{-c_{sf}}{M_{tr}} & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 \\
\frac{l_{r}k_{sr} - l_{r}k_{sf}}{J_{cw}} & \frac{l_{r}c_{sr} - l_{r}c_{sf}}{J_{cw}} & \frac{-l_{r}k_{sr}}{J_{cw}} & \frac{-l_{r}c_{sr}}{J_{cw}} & 0 & 0 & 0 \\
\end{bmatrix}\]

\[ \text{(9.3)} \]
\[
\begin{bmatrix}
\frac{dv_{cw}(t)}{dt} \\
\frac{da_{cw}(t)}{dt} \\
\frac{dv_{ir}(t)}{dt} \\
\frac{da_{ir}(t)}{dt} \\
\frac{dv_{fr}(t)}{dt} \\
\frac{da_{fr}(t)}{dt} \\
\frac{dv_{fr}(t)}{dt} \\
\frac{db_{cw}(t)}{dt} \\
\frac{db_{cw}(t)}{dt}
\end{bmatrix}
= [A] \times \begin{bmatrix}
v_{cw}(t) \\
a_{cw}(t) \\
v_{ir}(t) \\
a_{ir}(t) \\
v_{fr}(t) \\
a_{fr}(t) \\
\phi'_{cw}(t) \\
\phi''_{cw}(t) \\
\phi''_{cw}(t)
\end{bmatrix} + \begin{bmatrix}
0 & 0 \\
0 & 0 \\
k_{ir} & 0 \\
0 & k_{fr} \\
0 & 0 \\
0 & 0 \\
0 & 0 \\
0 & 0
\end{bmatrix} \times \begin{bmatrix}
v_{yr}(t) \\
v_{yr}(t)
\end{bmatrix} \quad (9.4)
\]

In order to explain the equations 9.3 and 9.4, some parts are discussed here. First of all, the equation determining \( \frac{da_{cw}(t)}{dt} \) is explained, see second row in equation 9.3 and 9.4. According to equation 9.4, \( \frac{da_{cw}(t)}{dt} \) is determined by the change in vertical forces that act on the mass of the carriage work. Changes in these forces are caused by:

\( v_{cw}(t) \): The vertical speed of the carriage work itself effects the forces that work in the suspension springs of the vehicle, \( \frac{dF}{dt} = -k_{sr}.v_{cw}(t) - k_{sf}.v_{cw}(t) \).

\( a_{cw}(t) \): The vertical acceleration of the carriage work effects the forces that work in the shock absorbers of the suspension of the vehicle, \( \frac{dF}{dt} = -c_{sr}.a_{cw}(t) - c_{sf}.a_{cw}(t) \).

\( v_{ir}(t) \): The vertical speed of the rear axle effects the force that is transferred to the carriage works by the rear suspension spring, \( \frac{dF}{dt} = k_{sr}.v_{ir}(t) \).

\( a_{ir}(t) \): The vertical acceleration of the rear axle effects the force that is transferred to the carriage works by the rear shock absorber, \( \frac{dF}{dt} = c_{sr}.a_{ir}(t) \).

\( v_{fr}(t) \): The vertical speed of the front axle effects the force that is transferred to the carriage works by the front axle suspension.
spring, \( \frac{dF}{dt} = k_s v_s(t) \).

\[ a_{uf}(t): \] The vertical acceleration of the front axle affects the force that is transferred to the carriage works by the front shock absorber, \( \frac{dF}{dt} = c_{sf} a_{uf}(t) \).

\[ \phi'_{cw}(t): \] The rotational speed of the carriage work effects forces in both the front and rear suspension springs, \( \frac{dF}{dt} = \phi'_{cw}(t). (I_r k_{sr} - I_r k_{sf}) \).

\[ \phi''_{cw}(t): \] The rotational acceleration of the carriage work effects forces in both the front and rear shock absorber, \( \frac{dF}{dt} = \phi''_{cw}(t). (I_r c_{sr} - I_r c_{sf}) \).

Adding up all the terms mentioned above explains the second row in the equations 9.3 and 9.4. By dividing all the mentioned \( \frac{dF}{dt} \)-values by the mass of the carriage work \( M_{cw} \), the change in vertical acceleration of the carriage work is known.

To explain how roughness will effect the movements of the three mass spring system the fourth row in the equations 9.3 and 9.4 is explained. In this row the value of \( da_{uf}(t)/dt \) is determined. For the explanation of the changes in the vertical acceleration of the rear axle the changes in the forces that act on the rear axle are of importance;

\[ v_{cw}(t): \] The vertical speed of the carriage work changes the force that acts in the suspension spring, \( \frac{dF}{dt} = k_s v_{cw}(t) \).

\[ a_{cw}(t): \] The vertical acceleration of the carriage work effects the forces that work in the front shock absorber, \( \frac{dF}{dt} = c_{sf} a_{cw}(t) \).

\[ v_{ir}(t): \] The vertical speed of the rear axle affects the forces in both the rear suspension spring and the spring representing the rear tire, \( \frac{dF}{dt} = -k_{sr} v_{ir}(t) - k_{sr} v_{ir}(t) \).

\[ a_{ir}(t): \] The vertical acceleration of the rear axle affects the force that is generated in the shock absorber, \( \frac{dF}{dt} = -c_{ir} a_{ir}(t) \).

\[ \phi'_{cw}(t): \] The rotational speed of the carriage works effects the force in the rear suspension springs, \( \frac{dF}{dt} = \phi'_{cw}(t). -I_r k_{sr} \).

\[ \phi''_{cw}(t): \] The rotational acceleration of the carriage works effects the force that is generated in the rear shock absorber, \( \frac{dF}{dt} = \phi''_{cw}(t). -I_r c_{sr} \).

To take into account the effects of roughness, the vertical speed at the bottom of the spring representing the rear tire has to be considered. This is done by means of the second matrix in equation 9.4. In the fourth row of this matrix one finds that the vertical speed of the bottom of the spring representing the rear tires effects the force in this spring and thus changes the force acting on the mass representing the rear tires and axle: \( \frac{dF}{dt} = v_{yr}(t) k_{sr} \).

The changes in the vertical acceleration of the mass representing the rear axle are now found by dividing all the terms discussed above by \( M_{ir} \). The fourth row of equation 9.4 is now explained.

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For the ease of writing, a vector \( \{z(t)\} \), a vector \( \{v_y(t)\} \) and a matrix \([B]\) are introduced, so that equation 9.4 becomes:

\[
\frac{d\{z(t)\}}{dt} = [A] \times \{z(t)\} + [B] \times \{v_y(t)\} \tag{9.5}
\]

Within the vehicle simulation it is assumed that the vehicle travels at a constant speed of \( v \) [m/s]. As a result the vectors \( \{z(t)\} \) and \( \{v_y(t)\} \) can be divided by \( v \):

\[
\frac{d\{z(t)\}}{dt} = \frac{[A]}{v} \times \frac{\{z(t)\}}{v} + \frac{[B]}{v} \times \frac{\{v_y(t)\}}{v} \quad \text{or} \tag{9.6}
\]

\[
\frac{d\{Z(t)\}}{dt} = [A] \times \{Z(t)\} + [B] \times \{V_y(t)\}
\]

The longitudinal profile heights are measured at constant intervals of "dl [m]", as a result \( dt \) is constant that equals \( dl/v \) [s]. To solve equation 9.6 the approach followed by the World Bank for the simulation of a quarter vehicle (International Roughness Index, IRI) is applied (38). For this approach the following matrices are to be determined:

\[
[ST] = \mathbf{I} + [A] \times dt + \frac{[A]^2 \times dt^2}{2!} + \frac{[A]^3 \times dt^3}{3!} + \ldots \tag{9.7}
\]

\[
[PR] = [A]^{-1} \times ([ST] - \mathbf{I}) \times [B] \tag{9.8}
\]

where:
- \( [I] \): identity matrix
- \([A], [B]\): earlier described matrices
- \( dt \): time used by simulated vehicle to travel the measuring distance 'dl' [s]
- \([ST]\): state transition matrix
- \([PR]\): particular response matrix

On the basis of the matrices \([ST]\) and \([PR]\) the vector \( \{Z(t)\} \) is now determined by:

\[
\{Z(t)\} = [ST] \times \{Z^*(t)\} + [PR] \times \{V_y(t)\} \tag{9.9}
\]
where:

\( \{Z(t)\} \): earlier described vector \( \{z(t)\} \), containing the vertical speeds and accelerations of the various masses and the rotational speed and acceleration of the carriage work, divided by the speed of the vehicle \( v \) \([\text{m/s}]\).

\( \{Z'(t)\} \): vector \( \{Z(t)\} \) at previous point on the longitudinal profile.

\( \{V_z(t)\} \): vector \( \{v_z(t)\} \), containing the vertical speeds with which the bottom side of the springs representing the tires are forced to move, divided by \( v \).

Since \( \{Z(t)\} \) can be determined it is possible to calculate the axle loads and vertical mass positions on the basis of this vector \( \{Z(t)\} \). By multiplying \( \{Z(t)\} \) by the speed of the vehicle, the vector \( \{z(t)\} \) is obtained. From this last vector the forces in the various parts of the vehicle model can be determined.

**fig 9.3** Dynamic forces in the model of a half vehicle.

For the dynamic forces shown in figure 9.3 the following equations hold:

\[
F_{str}(t) = a_{cw}(t) \times M_{cw} \times \frac{l_r}{l_f + l_r} + \frac{\phi''_{cw}(t) \times J_{cw}}{l_f + l_r} \tag{9.10}
\]

\[
F_{str}(t) = a_{cw}(t) \times M_{cw} \times \frac{l_f}{l_f + l_r} - \frac{\phi''_{cw}(t) \times J_{cw}}{l_f + l_r} \tag{9.11}
\]
\[ F_{kfr}(t) = a_r(t) \times M_{tr} + F_{str}(t) \]  
(9.13)

\[ F_{kfr}(t) = F_{str}(t) + (\phi_{cw}(t) \times l_f - v_{rf}(t) + v_{cw}(t)) \times c_f \]  
(9.14)

\[ F_{ksr}(t) = F_{str}(t) + (\phi_{cw}(t) \times l_r - v_{rm}(t) + v_{cw}(t)) \times c_r \]  
(9.15)

\[ F_{cfr}(t) = (\phi_{cw}(t) \times l_f + v_{rf}(t) - v_{cw}(t)) \times c_f \]  
(9.16)

\[ F_{cwr}(t) = (\phi_{cw}(t) \times l_r + v_{rm}(t) - v_{cw}(t)) \times c_r \]  
(9.17)

The vertical positions of the various masses in the three mass spring system can now be determined on the basis of the dynamic forces in the four springs. By dividing these forces by the stiffness of the spring in which they act, the compression of the springs is obtained. From these spring compressions of course the positions of the various masses can be determined.

In the previous discussion the development of the dynamic component of the forces that act in the vehicle model are discussed. It is clear that the forces that are present in the springs of the model have to be enlarged with the static forces, in order to come to the absolute forces that act in the vehicle model. For the static axle loads the following equations hold.

\[ F_{ssf} = g \times M_{cw} \times \frac{l_r}{l_f + l_r} \]  
(9.18)

\[ F_{ssr} = g \times M_{cw} \times \frac{l_f}{l_f + l_r} \]  
(9.19)

\[ F_{tsf} = F_{ssf} + g \times M_{tf} \]  
(9.20)

\[ F_{tsr} = F_{ssr} + g \times M_{tr} \]  
(9.21)

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Within the equations 9.10 to 9.21 the following symbols have been introduced:

- \( F_{str}(t), F_{str}(t) \): total dynamic compressive force in the front and rear suspension system respectively [N].
- \( F_{krf}(t), F_{krf}(t) \): dynamic compressive force in the front tires and rear tires respectively [N].
- \( F_{krf}(t), F_{krf}(t) \): dynamic compressive force in the front suspension spring and the rear suspension spring respectively [N].
- \( F_{csf}(t), F_{csf}(t) \): dynamic compressive force in the front shock absorber and the rear shock absorber respectively [N].
- \( F_{ssf}, F_{ssr} \): static compressive force in the front suspension system and the rear suspension system respectively [N].
- \( F_{tsf}, F_{tsr} \): total static compressive force in the front tires and the rear tires respectively [N].
- \( g \): gravitational acceleration (9.8 m/s\(^2\)) [m/s\(^2\)].

Given the previous equations the static and the dynamic component of both the front and the rear axle loads are known. The sum of both components of course gives the axle loads that are applied to the pavement.

### 9.2.2 Modelled vehicles

The vehicle model can of course be used to model any vehicle in which the dampers as well as the springs are linear. To get however realistic vehicle properties use is made of the properties of a two axled DAF-truck described elsewhere (39).

- \( M_{cw} \): 5,500 kg,
- \( J_{cw} \): 9,640 kg m\(^2\),
- \( M_{fr} \): 800 kg,
- \( M_{rr} \): 1,300 kg,
- \( k_{sf} \): 430,000 N/m,
- \( k_{sr} \): 700,000 N/m,
- \( c_{sf} \): 30,000 Ns/m,
- \( c_{sr} \): 30,000 Ns/m,
- \( k_{fr} \): 1,500,000 N/m,
- \( k_{rr} \): 4,000,000 N/m,
- \( l_{f} \): 1.02 m and
- \( l_{r} \): 2.18 m.

If a vehicle is loaded only four of the given properties will be effected, i.e.: \( M_{cw}, J_{cw}, l_{f} \) and \( l_{r} \). In order to be able to determine the values of these vehicle properties for a loaded DAF-truck additional information about the
position of the loadplatform of the vehicle is needed. This information is however not given in the report mentioned above.

In the next example the properties of a truck, when it is loaded in such a way that the static rear axle load becomes 100 kN, are determined. The assumptions that had to be made because of the lack of information about the dimensions of the vehicle are discussed in the example.

The axle loads of the unloaded truck have to be determined first. Given the position of the centre of gravity of the 5,500 kg carriage work the front axle is loaded by \( (5,500 \times \frac{l_f}{l_f + l_r} = 3,747 \text{ kg}) \), so that the rear axle is loaded by 1,753 kg. Adding the weight of the axles the static load on the pavement is known, for the front axle this becomes 4,547 kg and for the rear axle this becomes 3,053 kg.

To get a static 100 kN rear axle load the rear wheels should carry a \( (100,000/g = 100,000/9.8 = 10,204 \text{ kg load}) \). The payload should thus add an additional 7,151 kg load to the rear axle. The magnitude of the load that will deliver this additional weight to the rear axle depends on the position of the centre of gravity of the payload. For this research it is estimated that the load area of the truck starts 0.5 m behind the front axle and has a total length of 3.7 m so that it ends 1 m behind the rear axle. It is furthermore assumed that the payload is equally divided over the load area so that the centre of gravity of the payload is located 2.35 m behind the front axle. This means that 2.35/3.2 of the payload is transferred to the rear axle, a 9,737 kg payload thus results in a 10,204 kg rear axle load.

Now the magnitude of the payload is known, the mass of the carriage work including payload \( "M_{cw}" \) is known too and becomes \( (5,500 + 9,737 = 15,237 \text{ kg}) \).

The next step is to calculate the position of the centre of gravity of the carriage work including payload.

\[
l_f = l_{f_{cw}} + \left( l_f - l_{f_{cw}} \right) \times \frac{M_f}{M_{cw}} \tag{9.22}
\]

where:

- \( l_f \): horizontal distance between the front axle and the centre of gravity of the carriage work including payload, if any [m]
- \( l_{f_{cw}} \): horizontal distance between the front axle and the centre of gravity of the unloaded carriage work [m]
- \( l_r \): horizontal distance between the front axle and the centre of gravity of the payload [m]
- \( M_f \): mass of the payload [kg]
- \( M_{cw} \): mass of the carriage work including payload, if any [kg]
Given equation 9.22, \( l_i \) becomes 1.87 m and \( l_f \) becomes 1.33 m, since the distance between the rear and front axle is 3.2 m.

The only vehicle property of the loaded truck that still is to be determined is the moment of inertia of the carriage work including payload \( J_{cw} \). Hereto first of all the moment of inertia of the load itself has to be determined. Assuming the load has a width of 2 m and a specific gravity of 1000 kg/m\(^3\) the height of the load becomes \((9,737/(3.7 \times 2 \times 1000)) = 1.316\) m. The moment of inertia of the load \( J_l \) now becomes \(((1/12)\times(2^2+1^2)\times M_l = 12,514\) kg.m\(^2\). Given the distance between the centre of gravity of the payload and the centre of gravity of the carriage work including the payload, \( d_i \), and the distance between the centre of gravity of the carriage work and the centre of gravity of the carriage work including the payload, \( d_{cwa} \), the moment of inertia of the carriage work including the load \( J_{cw} \) becomes.

\[
J_{cw} = J_l + J_{cwa} + M_i \times d_i^2 + M_{cwa} \times d_{cwa}^2
\]

(9.23)

where:
- \( d_i \): distance between the centre of gravity of the payload and the centre of gravity of the carriage work including payload [m]
- \( d_{cwa} \): distance between the centre of gravity of the unloaded carriage work and the centre of gravity of the carriage work including payload [m]
- \( J_{cw} \): moment of inertia of the carriage work including payload [kg.m\(^2\)]
- \( J_l \): moment of inertia of the payload [kg.m\(^2\)]
- \( J_{cwa} \): moment of inertia of the unloaded carriage work [kg.m\(^2\)]

Only considering the horizontal dimensions of the truck, \( d_i \) becomes \((2.35-1.87=)\) 0.48 m and \( d_{cwa} \) becomes \((1.87-1.02=)\) 0.85 m. \( J_{cw} \) now becomes \((12,513+9,640+3,974+2,244 =)\) 28371 kg.m\(^2\).

Notice that the difference in vertical position of the centre of gravity of the payload and the centre of gravity of the unloaded carriage work is neglected. This is done since the vertical position of the centre of gravity of the carriage work of the unloaded truck is not known. However, one has to realize that the effects of considering the vertical distance between the centre of gravity of the carriage work and the centre of gravity of the payload on \( J_{cw} \) remain limited when compared to the effects of the width and the specific gravity of the payload itself on \( J_{cw} \). If the vertical distance between the two centres of gravity for instance equals half the difference in horizontal direction, then \( J_{cw} \) should be enlarged with \((0.5^2\times(3,974+2,244) =)\) 1,554 kg.m\(^2\) which is an increase of only 5.5%.

Much larger effects are a result of the properties of the payload. If the load for instance consists of 10 layers of 80 mm concrete blocks then the height of the load would be 0.8 m. Since concrete blocks have a specific
gravity of about 2400 kg/m³, the length of the load will now be (9,737/2x2400x0.8) = 2.536 m, so that $J_e$ becomes 5,738 kg.m². Assuming the concrete blocks are placed in the middle of the load area the centre of gravity of the load is not changed so that it will again lead to a 100 kN rear axle load. $J_{ew}$ now is (12,513-5,738 =) 6,775 kg.m² smaller than discussed earlier, which is 23.9%. This shows that the properties of the payload itself are much more important than the unknown vertical dimensions of the vehicle.

On the basis of the previous discussion the following vehicle properties change as a function of the static rear axle load, see table 9.1. Within this table a truck with a 29.92 kN static rear axle load is present. This truck refers to the DAF-truck without any payload.

<table>
<thead>
<tr>
<th>Static rear axle load [kN]</th>
<th>empty vehicle</th>
<th>loaded vehicles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>29.92</td>
<td>80</td>
</tr>
<tr>
<td>$l_r$ [m]</td>
<td>1.02</td>
<td>1.763</td>
</tr>
<tr>
<td>$l_r$ [m]</td>
<td>2.18</td>
<td>1.437</td>
</tr>
<tr>
<td>$M_{cw}$ [kg]</td>
<td>5,500</td>
<td>12,459</td>
</tr>
<tr>
<td>$J_{ew}$ [kg.m²]</td>
<td>9,640</td>
<td>23,525</td>
</tr>
</tbody>
</table>

*table 9.1* Vehicle properties that change as a function of the static rear axle load.

### 9.2.3 Results of a vehicle simulation

The vehicle model discussed in the previous section can now be used to simulate a vehicle travelling over a pavement. To get insight in the principles of such a simulation a simple example is discussed in this paragraph. The example considers the behaviour of a vehicle travelling a completely planar pavement in which a 150 mm high speed reducing bump with a length of 3 m is present. The vehicle taken under consideration is a loaded DAF-truck with a 100 kN static rear axle load that travels at a speed of 10 km/h.

In figure 9.4 the speed bump is presented, in the same figure the positions of the carriage work above the front and rear axle are shown. Also the positions of the centre of gravity of the masses representing the front and rear axles are shown.
fig 9.4  Movements of various parts of the vehicle when travelling over a speed bump.

As is shown by figure 9.4 the carriage work above the rear axle is forced down when the front wheels of the vehicle are forced upward by the speed hump. This is a result of mass inertia of the carriage work including payload. If the front of the carriage work goes up the rear wants to go down in order to fix the vertical position of the centre of gravity of the carriage work including payload. The rear suspension springs are as a result compressed and thus push the rear part of the carriage work up again. Before the rear wheels hit the speed bump, the movements of the rear part of the carriage work as a result are more or less sine shaped.

Similar sine shaped movements are found when the vehicle passed the speed bump. Due to the shock absorbers the amplitude of these waves decrease when the vehicle travels along.

The bottom part of the figure shows the movements of the centre of gravity of both the rear and the front axle. These movements show similar shapes as are found for the carriage work. As a result of the large stiffness of the tires the amplitudes found for the vertical movements of the tires remain much smaller.

It is clear that the deformation of the tires combined with their stiffness will give the dynamic component of the axle loads. By adding the static axle loads to these dynamic components the total dynamic axle loads are obtained. In figure 9.5 the development of these axle loads are plotted together with the longitudinal profile.
Figure 9.5 shows that the rear axle load will show an increase at the moment the front axle hits the speed bump. This of course is a direct result of the fact that the front part of the vehicle is pushed up by the speed bump as discussed earlier. Again it is found that when the vehicle has travelled the complete speed bump sine shaped axle load developments are found that show a decreasing amplitude when the vehicle travels along. Again this decrease in the amplitudes is a result of the shock absorbers.

9.3 SIMULATION OF VEHICLE INDUCED LONGITUDINAL UNEVENNESS

9.3.1 Pavement behaviour

Longitudinal unevenness induced by vehicles is a result of the rut depth not being constant over the length of the road. As discussed in the previous paragraph a vehicle that travels over a longitudinal profile that is not perfectly planar will show dynamic reactions to the roughness of the pavement surface. As a result of these reactions the dynamic axle loads that are introduced by the vehicle will vary over the chainage. In order to simulate the effects of the varying axle loads on the development of a rut depth varying with the chainage, the rut development in a pavement as a function of the magnitude of
the axle loads and the number of load repetitions has to be known.

In section 8.5 the rutting behaviour of concrete block pavements was discussed. In this section it was explained that development of ruts in concrete block pavement as a function of \( N_s \), can be described by equation 9.24, which equals equation 8.12. The number of equivalent standard wheel loads "\( N_{st} \)" that is caused by a single passage of a dynamic axle load can be determined by means of equation 9.25, which closely resembles equation 8.13. The development of ruts in a pavement is now known as a function of both the number of load repetitions and the magnitude of the dynamic axle loads.

\[
RD_r = a_p \left( \frac{N_{st}}{1000} \right)^{b_p} - c_p \left( e^{d_p \frac{N_{st}}{1000}} - 1 \right) \tag{9.24}
\]

\[
N_{st} = \left( \frac{L_i}{L_{st}} \right)^{m_i} \times \left( \frac{\sigma_{c,i}}{\sigma_{c,st}} \right)^{m_r} \tag{9.25}
\]

where:

- \( a_p \), \( c_p \): pavement parameters [mm]
- \( b_p \), \( d_p \), \( m_i \), \( m_r \): pavement parameters [-]
- \( L_{st} \): standard wheel load [kN]
- \( L_i \): actual dynamic wheel load applied to the pavement [kN]
- \( \sigma_{c,st} \): standard wheel load contact pressure [kPa]
- \( \sigma_{c,i} \): actual wheel load contact pressure [kPa]
- \( N_{st} \): number of equivalent standard wheel load repetitions [-]
- \( RD_r \): relative rut depth or rut depth underneath a 1.2 m straight edge [mm]

On the basis of the equations 9.24 and 9.25 the development of the relative rut depth in a pavement is described. From this relative rut depth "\( RD_r \)" the absolute rut depth "\( RD_a \)" can be calculated. The absolute rut depth is a factor "RP" larger than \( RD_r \), by multiplying \( RD_r \) with \( RP \) the absolute rut depth is thus obtained, see equation 8.15.

All the pavement parameters that are required can by determined by using the rutting performance model discussed earlier in chapter 8, and are not discussed here.

It is stated here that the rutting model considers the wheel load magnitude while the vehicle model gives insight into the dynamic axle loads. In this research it is simply assumed that a wheel load equals half the axle load.
9.3.2 Interaction between pavement and vehicle

As discussed a vehicle will react to pavement roughness by subjecting the pavement to dynamic axle loads that vary over the length of the pavement. A pavement on its turn develops ruts, the magnitude of these ruts depends on both the number of load repetitions and the magnitude of the axle loads. The combination of these two facts leads to vehicle-introduced rut depth varying with the chainage. Of course such a varying rut depth will have its effects on the development of roughness in the pavement, see figure 9.6.

Figure 9.6 shows three lines. The upper line represents the initial longitudinal profile of a fictitious pavement. The middle 1/3 part of the pavement is perfectly planar and shows no roughness at all. The two outer parts of the pavement however show firm roughness. After the pavement has been trafficked for some time a rut depth varying with the chainage has developed. In the first part of the pavement no ruts have developed at all. The last 2/3 part of the pavement however shows an absolute rut depth that is varying with the chainage.

![Diagram showing initial longitudinal profile, absolute rut depth varying with chainage, and longitudinal profile in middle of rut.](image)

**fig 9.6** Principal of the effects of a varying absolute rut depth on the longitudinal profile in the wheeltracks.

The bottom line in figure 9.6 represents the longitudinal profile in the wheeltrack (centre of the rut). For the first part of the pavement this longitudinal profile equals the initial longitudinal profile since the absolute rut depth in the first part of the pavement equals nil.

In the second part of the pavement the longitudinal profile in the
wheeltrack is completely determined by the absolute rut depth varying over the chainage. The last part of the pavement shows initial roughness and an absolute rut depth varying over the chainage. The longitudinal profile in the wheeltrack in this part of the pavement is thus determined by both the initial longitudinal profile and the varying absolute rut depth.

The interaction between a vehicle and the longitudinal profile of a pavement can be simulated on the basis of the vehicle model and the rutting behaviour of the pavement, both discussed earlier. Hereto the considered vehicle is first simulated travelling over the initial longitudinal profile of the pavement in question. Each time this vehicle simulation is completed for the next point in the longitudinal profile the additional absolute rut depth induced by the dynamic front axle load is computed, see figure 9.7, and is hereafter subtracted from the profile height at the front wheels of the vehicle (without actually lowering these wheels). The rear axle of the vehicle will as a result travel over a longitudinal profile in which the effects of the varying absolute rut depth induced by the front axle is already present.

![Diagram](image)

**fig 9.7**  Simplified principal of the repeated vehicle simulation.

The additional absolute rut depth introduced by a single axle load repetition is determined by computing the number of equivalent standard wheel loads that the dynamic axle load introduces, see equation 9.25. The contact pressure is assumed not to depend on the dynamic wheel load magnitude and does thus not change. The number of equivalent standard
wheel loads introduced by the dynamic wheel load is then added to the number of standard wheel loads applied to the pavement, on this point in the longitudinal profile, by previous vehicles, if any. On the basis of the total number of equivalent standard wheel loads the absolute rut depth introduced by all previous vehicles, including the wheel load under consideration can now easily be determined, see equation 9.24. This absolute rut depth is then subtracted from the initial profile height to take into account the effects of the wheel load repetition under consideration.

Similarly the additional absolute rut depth induced by the dynamic rear axle load is computed. This additional absolute rut depth is then subtracted from the profile height at the rear wheels (without actually lowering these wheels). The following vehicle will thus travel over a longitudinal profile in which the effects of the varying rut depth induced by the current vehicle are taken into account.

This process is repeated until the vehicle reaches the end of the longitudinal profile in question.

When the simulated vehicle has reached the end of the longitudinal profile the simulation is repeated on the basis of the new longitudinal profile. When the effects of enough vehicles on the development of the longitudinal profile have been determined the process is stopped. The development of longitudinal unevenness in the pavement due to the repeatedly simulated vehicle is then known.

To give insight into the results of the described repeated vehicle simulation the perfectly planar pavement with the speed bump is considered again. The vehicle travelling over this bump is again the DAF-truck that has a static rear axle load of 100 kN. The static front axle load of this vehicle is 69.9 kN. The DAF-truck is simulated travelling at a speed of 10 km/h.

It is assumed that the concrete block pavement only has a 1000 mm thick Zaanweg sand sub-base over a 60 MPa subgrade and that the standard deviation of lateral wander is 200 mm. The rut development properties of this pavement have been determined for the case that the pavement is only trafficked by 50 kN wheel loads with a 700 kPa contact pressure: $a_c=5.560$ mm, $b_c=0.2012$, $c_c=0$ mm, $d_c=0$, $m_i=10.95$, $m_c=3.98$, $L_{st}=50$ kN, $\sigma_{c,m}=700$ kPa and $RP_c=1.64$.

This implies that almost 140,000 equivalent standard wheel loads are required to induce a 15 mm relative rut depth. Considering the static axle loads of the DAF-truck under consideration the static wheel loads become 50 kN and 34.95 kN for the rear and front wheels respectively. Assuming that all wheel loads are applied to the pavement with a 700 kPa contact pressure this implies that more than 136,000 vehicle passages are needed in order to come
to a 15 mm relative rut depth in the case that the dynamical effects are not taken into account.

In figure 9.8 the dynamic axle loads are shown that develop when the vehicle travels over the speed bump at 10 km/h. It is clear that these dynamic axle loads are far from constant so that a rut depth varying with chainage will develop and affect the longitudinal profile.

\[
N_{\text{vehicle}} = 0
\]

\[
\text{axle load [kN]}
\]

\[
\text{chainage [m]}
\]

\[
\text{profile height [m]}
\]

![Dynamic axle loads of the vehicle when travelling over the undeformed longitudinal profile of the speed bump.](image)

**Fig 9.8** Dynamic axle loads of the vehicle when travelling over the undeformed longitudinal profile of the speed bump.

Figure 9.9 shows the development of the longitudinal profile as a result of the repeated trafficking by dynamic wheel loads and the characteristics of the pavement.

The dynamic axle loads that are introduced to the pavement show a relatively large value directly after the speed bump, see figure 9.8. As a result the pavement at this point is relatively heavily loaded and will thus show a relatively large rut depth. Directly after the speed bump the longitudinal profile will thus develop a depression as a result of repeated vehicle passages, see figure 9.9.

By considering figure 9.9 some other properties of traffic induced longitudinal unevenness become clear. The depression that develops directly after the speed bump of course is a form of longitudinal unevenness. Since longitudinal unevenness causes dynamic vehicle responses, the section in which the dynamic wheel loads strongly differ from the static wheel loads will lengthen due to the development of the depression during trafficking. Of course this, in its turn, implies that the length over which traffic induced longitudinal unevenness will develop increases during trafficking. In figure
9.9 this is shown by comparing the longitudinal profiles obtained after 1,361 simulated vehicles \( (N_{\text{vehicle}} = 1,361) \) and the profile given for \( N_{\text{vehicle}} = 136,088 \).

![Diagram showing longitudinal profiles](image)

**Fig 9.9** Development of the longitudinal profile in the wheeltrack around the speed bump due to repeated vehicle passages.

Another property of the interaction between pavement and vehicles is that longitudinal unevenness is slowly moving forward. Again this can be seen in figure 9.9. The depressions that are introduced by the vehicles slowly move in the direction of traffic.

![Diagram showing dynamic axle loads](image)

**Fig 9.10** Dynamic axle loads of the vehicle around the speed bump at \( N_{\text{vehicle}} = 136,088 \).
In figure 9.10 the dynamic axle loads of the vehicle at $N_{\text{vehicle}} = 136,088$ are presented. By comparing figure 9.10 with 9.8 it is shown that the section in which dynamic axle loads are introduced lengthens during trafficking, as discussed earlier. At the far right side of the figure 9.10, where the chainage equals 20 m, dynamic effects are still clearly visible which is not so much the case in figure 9.8.

Comparison of figures 9.8 and 9.10 furthermore shows that maximum dynamic axle loads that develop when the vehicle travels over the longitudinal profile decrease during trafficking. This is a result of the traffic induced change in shape of the speed bump. The front side of the bump clearly becomes more rounded whereas the drop in profile height at the end of the speed bump decreases during trafficking, see figure 9.9.

In this research traffic induced longitudinal unevenness is explained by considering the absolute rut depth varying with the chainage. In the figures 9.11 and 9.12 the rut depth is shown. In figure 9.11 the development of the relative rut depth is shown while figure 9.12 shows the development of the absolute rut depth. Figure 9.12 thus shows the differences between the initial longitudinal profile heights and the profile heights of the longitudinal profiles as they develop during trafficking.

![Graph](image)

**fig 9.11** Development of the relative rut depth around the speed bump.

Both the figures show the development of a strong depression at the beginning of the speed bump. In the previous plots this depression was not clearly reflected due to the relatively large effect of the initial longitudinal profile on the longitudinal profiles that follow from it. By considering the
figures 9.11 and 9.12 it becomes clear that depressions develop at those spots where large axle loads occur.

**Fig 9.12** Development of the absolute rut depth around the speed bump.

### 9.4 MODEL OF THE DEVELOPMENT OF VEHICLE INDUCED LONGITUDINAL UNEVENNESS

In the previous section the simulation of the development of vehicle induced longitudinal unevenness was discussed. The described repeated vehicle simulation however has the strong disadvantage that it is very time consuming. Each vehicle travelling over a pavement has to be simulated in order to take into account its effects on the development of longitudinal unevenness. A realistic simulation as a result can take hours or even days, depending on the characteristics of the computer.

Therefore a simplified model has been developed which requires much less computation time. This model is described in this section.

#### 9.4.1 The model

**9.4.1.1 Vehicle reactions**

Vehicles that have a linear behaviour show linear reactions to longitudinal unevenness. This means that if longitudinal unevenness becomes twice as severe the vehicle response will also become twice as intensive. For
the sake of simplicity, it is assumed in this research program, that all vehicles in traffic have a linear behaviour, i.e. linear springs and shock absorbers.

Of course the reactions of any vehicle to a longitudinal profile depend on the properties of the vehicle and the vehicle speed. For the development of a model explaining the development of longitudinal unevenness as a result of rut depth varying with chainage, it is assumed that traffic consists of different groups of vehicles. The vehicles in one group all have the same properties and travel at the same speed.

Dynamic axle loads are build up from two components, a static component "F_s," and a dynamic component "F_d". The value of F_s is constant and equals the static axle load. If a longitudinal profile shows unevenness F_d will vary with the chainage.

If longitudinal unevenness consists of a perfect sine the reactions of a vehicle with a linear behaviour will also show a perfect sine. The wavelength of the sine of the vehicle response equals the wavelength of the longitudinal unevenness, both sines however show a phase difference. The amplitude of the vehicle response is linearly dependent on the amplitude of unevenness, see also figure 9.13.

![Figure 9.13](image)

*Figure 9.13* Relation between dynamic axle loads and sine waved roughness.

The model that explains the development of longitudinal unevenness as a result of a rut depth varying with chainage considers longitudinal unevenness...
with different wavelengths present in the longitudinal profile individually. For each individual wavelength longitudinal unevenness is now a pure sine.

The different vehicle groups in traffic are indicated by the letter "i". For the vehicles within such a group the following equations hold.

\[
A_{u_{\lambda,i}}[N_{st}] = Au_{\lambda}[N_{st}] \times ff_{\lambda,i} \\
A_{fr_{\lambda,i}}[N_{st}] = Au_{\lambda}[N_{st}] \times fr_{\lambda,i}
\] (9.26)

where:
\[
Au_{\lambda}[N_{st}]: \quad \text{Amplitude of unevenness with wavelength } \lambda \text{ after } N_{st} \text{ equivalent standard axle load repetitions [mm]}
\]
\[
A_{fr_{\lambda,i}}[N_{st}], A_{ff_{\lambda,i}}[N_{st}]: \quad \text{Amplitude of dynamic component of the rear and front axle load respectively of vehicle } i \text{ with a wavelength } \lambda \text{ after } N_{st} \text{ equivalent standard axle load repetitions [kN]}
\]
\[
fr_{\lambda,i}, ff_{\lambda,i}: \quad \text{Parameters relating the amplitude of the dynamic components of the rear and front axle load of vehicle } i \text{ respectively to the amplitude of unevenness, both with a wavelength } \lambda \text{ [kN/mm]}
\]

### 9.4.1.2 Simplified concrete block pavement behaviour

In order to prevent repeated vehicle simulation it is needed to simplify, i.e. linearize, the effects of axle load variations on the rut development around the rut development induced by the static loads of the vehicles in traffic. The equations 9.27, 9.28 and 9.29 give the actual pavement behaviour considering the development of ruts.

\[
RD_{\nu}[N_{st}] = RP \times \left( a_p \left( \frac{N_{st}}{1000} \right)^{b_p} + c_p \left( \frac{N_{st} - d_p}{1000} \right) - 1 \right) \] (9.27)

with

\[
Neq = \frac{1}{100} \sum_{i=1}^{nu} \left( \frac{L_i}{AL_{st}} \right)^{m_i} \times \left( \frac{L_{r_{i,c}}}{\sigma_{c,st}} \right)^{m_i} \times \left( \frac{Lr_{i,c}}{AL_{st}} \right)^{m_i} \times \left( \frac{C_{r_{i,c}}}{\sigma_{c,st}} \right)^{m_i} \] (9.28)

and

\[
Ne = N \times Neq
\] (9.29)
where:

\( RD_a[N_{st}] \): absolute rut depth after \( N_{st} \) standard axle load repetitions [mm]

\( a_p, c_p, b_p, d_p, m_i, m_c, RP \): pavement properties [mm]

\( AL_{st}, L_f, L_r \): standard axle load which equals 2 \( \times \) \( L_{st} \) [kN]

\( N_{st} \): number of equivalent standard axle loads [-]

\( \%_i \): percentage of vehicles "i" in traffic [-]

\( vnu \): number of different vehicles (either in speed or in properties) in the traffic [-]

\( N_{eq} \): number of equivalent standard axle load repetitions introduced by a single fictitious vehicle that combines the properties of the different vehicles in traffic [-]

\( N \): number of vehicle passages [-]

As stated earlier the behaviour of the pavement as given above has to be linearized around the behaviour that is caused by the static axle loads of the vehicles in traffic. This is done by considering the following derivatives.

\[
\frac{dRD_a[N_{st}]}{dL_f} = \frac{dRD_a[N_{st}]}{dN_{st}} \times \frac{dN_{st}}{dL_f}
\]

\[
= \frac{RP}{1000} \left( a_p b_p \left( \frac{N_{st}}{1000} \right)^{b_p-1} + c_p d_p e^{\left( \frac{d_p N_{st}}{1000} \right)} \right) \times \frac{\%_i L_f}{N_{eq} \times \frac{m_i}{AL_{st}}} \left( \frac{\sigma_{c,i}}{\sigma_{c,st}} \right)^{m_i} \frac{1}{AL_{st}}
\]  \tag{9.30}

and

\[
\frac{dRD_a[N_{st}]}{dL_f} = \frac{RP}{1000} \left( a_p b_p \left( \frac{N_{st}}{1000} \right)^{b_p-1} + c_p d_p e^{\left( \frac{d_p N_{st}}{1000} \right)} \right) \times \frac{L_r}{100} \times \frac{\%_i}{m_i} \left( \frac{\sigma_{c,i}}{\sigma_{c,st}} \right)^{m_i} \frac{1}{AL_{st}}
\]  \tag{9.31}

For the further development of the model explaining the progress of longitudinal unevenness, the average effects of axle load variations on rutting over a certain number of equivalent standard axle load repetitions, \( N_{st,1} \) up to \( N_{st,2} \), have to be known. For the front axles of the vehicles equation 9.32 is
now found. The effects of rear axle load variations are described by equation 9.33.

$$\frac{dRD_a[N_{st,1}, N_{st,2}]}{dL_f_i} = \left( \frac{dRD_a[N_{st,2}]}{dL_f_i} - \frac{dRD_a[N_{st,1}]}{dL_f_i} \right) \frac{Neq}{N_{st,2} - N_{st,1}}$$  (9.32)

$$\frac{dRD_a[N_{st,1}, N_{st,2}]}{dL_r_i} = \left( \frac{dRD_a[N_{st,2}]}{dL_r_i} - \frac{dRD_a[N_{st,1}]}{dL_r_i} \right) \frac{Neq}{N_{st,2} - N_{st,1}}$$  (9.33)

Now the effects of variations in both the front and rear axle loads on rut development are known. Since the pavement behaviour is linearized around the behaviour as a result of the static axle loads the superposition principle is valid.

In figure 9.14 the rut depth introduced by a single repetition of a front axle load of vehicle group "i" is considered. As indicated in the figure the figure refers to the average rutting behaviour as present between $N_{st,1}$ and $N_{st,2}$.

The upper plot gives the sine of unevenness with a wavelength \( \lambda \) at $N_{st,1}$ equivalent standard load repetitions. The middle plot gives the response of the front axle of the vehicles in group "i" to this sine of unevenness. The bottom plot gives the sine in the rut depth as a result of the front axle response and the rutting behaviour at that moment in $N_{st}$.

![Fig 9.14](image)

**Fig 9.14**  Simplified interaction between longitudinal profile and axle load.

The sine of rut depth introduced by the front axle of a vehicle of course alters the longitudinal profile to a small extent. The sine of longitudinal
unevenness that is present after the front axle has passed can now be determined by adding the sine of unevenness and the sine of the rut depth variations. This is shown in principle in figure 9.15.

\[
\begin{align*}
\Delta f_{\lambda,i} & \times 2 \pi / \lambda \\
\Delta f_{\lambda,i} & \times 2 \pi / \lambda \\
A_{\lambda} \{N_{st1}\} & \times \\
A_{\lambda} \{N_{st1}\} & \times \\
C_{\lambda,i} \{N_{st1,2}\} & \\
C_{\lambda,i} \{N_{st1,2}\} & \\
\end{align*}
\]

**fig 9.15**  Principle of adding up the sine of unevenness and the sine of rut depth variations caused by the front axle.

On the basis of some goniometry, to a large extent shown in figure 9.15, the following equations can be derived. These equations give the effects of a front wheel passage on the amplitude of longitudinal unevenness with a certain wavelength.

\[
A_{\lambda} \{N_{st1}\} + \left( \frac{LF_i}{AL_{st}} \right)^m_i \left( \frac{\alpha f_{ci}}{\alpha_{c,ST}} \right)^m_c = A_{\lambda} \{N_{st1}\} \times \\
\times \sqrt{1 + C_{\lambda,i} \{N_{st1,2}\}^2 + S_{\lambda,i} \{N_{st1,2}\}^2}
\]

\[
C_{\lambda,i} \{N_{st1,2}\} = f_{\lambda,i} \cos \left( \frac{\Delta f_{\lambda,i}}{\lambda} 2 \pi \right) \frac{dRD_{a} \{N_{st1,2}\}}{dL_{f_i}}
\]

\[
S_{\lambda,i} \{N_{st1,2}\} = f_{\lambda,i} \sin \left( \frac{\Delta f_{\lambda,i}}{\lambda} 2 \pi \right) \frac{dRD_{a} \{N_{st1,2}\}}{dL_{f_i}}
\]

where:

- \( \Delta f_{\lambda,i} \): shift of the sine of rut depth introduced by the front axle of the vehicles in group "i" and the sine of longitudinal unevenness [m]

The rear axle will also affect the longitudinal profile. Of course the profile encountered by the rear axle will already show the effects of the front axle. In figure 9.16 it is shown how the effects of the varying rut depth
induced by the rear axle load of a vehicle in group "i" are taken into account.

\[ \text{fig 9.16} \quad \text{Principle of adding up the sine of unevenness and the sine of rut depth variations caused by both the front and rear axle.} \]

On the basis of some goniometry the following equations are derived for taking into account the effects of the rut depth induced by a rear axle load passage.

\[ Au_\lambda[N_{st},1 + \left( \frac{L_f}{AL_{st}} \right)^m + \left( \frac{L_r}{AL_{st}} \right)^n + \left( \frac{\sigma}{\sigma_{st}} \right)^p + \frac{\sigma r}{\sigma_{st}} \right] \]

\[ Au_\lambda[N_{st},1] \times \]

\[ \times \sqrt{\left[ 1 + C_f[N_{st},1,N_{st},2] + C_r[N_{st},1,N_{st},2] \right]^2 + \left( S_f[N_{st},1,N_{st},2] + S_r[N_{st},1,N_{st},2] \right)^2} \]

\[ (9.37) \]

\[ C_{r,\lambda}[N_{st},1,N_{st},2] = \sqrt{\left( 1 + C_f[N_{st},1,N_{st},2] \right)^2 + S_f[N_{st},1,N_{st},2]^2} \times \]

\[ \times \frac{\Delta r_{\lambda,i} \cos \left( \frac{\Delta L_{\lambda,i}}{\lambda} \right)}{\frac{\Delta L_{\lambda,i}}{\lambda} + \tan \left( \frac{S_f[N_{st},1,N_{st},2]}{1 + C_f[N_{st},1,N_{st},2]} \right)} \frac{dR}{dL} \]

\[ (9.38) \]
\[ S_{r_{\lambda,i}}[N_{st1},N_{st2}] = \sqrt{(1 + Cf_{\lambda,i}[N_{st1},N_{st2}])^2 + Sr_{\lambda,i}[N_{st1},N_{st2}]}^2 \times \]
\[ \times f_{r_{\lambda,i}} \sin \left( \frac{\Delta r_{\lambda,i}^2 2\pi}{\lambda} + \arctan \left( \frac{Sr_{\lambda,i}[N_{st1},N_{st2}]}{1 + Cf_{\lambda,i}[N_{st1},N_{st2}]} \right) \right) \frac{dRD_i[N_{st1},N_{st2}]}{dLr_i} \]

(9.39)

where:
\[ \Delta r_{\lambda,i} \] shift of the sine of rut depth introduced by the rear axle of the vehicles in group "i" and the sine of longitudinal unevenness [m]

Given the equations 9.34 to 9.39 the effects of one passage of a fictitious vehicle in which all vehicles in traffic are taken into account become:

\[ Au_{\lambda}[N_{st1} + Neq] = Au_{\lambda}[N_{st1}] \times \]
\[ \times \sqrt{(1 + Cfr_{\lambda,i}[N_{st1},N_{st2}])^2 + Sfr_{\lambda,i}[N_{st1},N_{st2}]}^2 \]

(9.40)

\[ Cfr_{\lambda,i}[N_{st1},N_{st2}] = \sum_{i=1}^{n1} (Cf_{\lambda,i}[N_{st1},N_{st2}]) \]

(9.41)

\[ Sfr_{\lambda,i}[N_{st1},N_{st2}] = \sum_{i=1}^{n1} (Sr_{\lambda,i}[N_{st1},N_{st2}]) \]

(9.42)

Since there might be more than one passage of the fictitious vehicle between \( N_{st1} \) and \( N_{st2} \), the amplitude of unevenness at \( N_{st2} \) becomes:

\[ Au_{\lambda}[N_{st2}] = Au_{\lambda}[N_{st1}] \times \]
\[ \times \sqrt{(1 + Cfr_{\lambda,i}[N_{st1},N_{st2}])^2 + Sfr_{\lambda,i}[N_{st1},N_{st2}]}^2 \frac{1}{N_{st2} - N_{st1}} \]

(9.43)

9.4.2 Determination of the model parameters

The described model, explained in the earlier paragraphs, is based on explaining the effects of traffic on longitudinal unevenness per wavelength. In order to use the model the amplitudes of unevenness have to be known as a function of the wavelength. The initial amplitudes of longitudinal unevenness "\( Au_{\lambda}[0] \)" are determined on the basis of a log-log relation between wavelength and the power density of longitudinal unevenness (40, 41). Given the value of
the power density at a wavelength of 5m "A(5m)" and the angle "ang" of the line, the amplitudes of the following wavelengths are now determined.

$$\lambda = \frac{\lambda_{\text{max}}}{\text{int}}$$  \hfill (9.44)

These amplitudes equal.

$$A_{u_{1}}[0] = \sqrt{A5m \left( \frac{\lambda_{\text{max}}}{5 \times \text{int}} \right)^{\text{ang}}} \times \left( \frac{\lambda_{\text{max}}}{\text{int}} - \frac{\lambda_{\text{max}}}{\text{int}+1} \right) \times 2$$  \hfill (9.45)

where:

- $\lambda_{\text{max}}$: largest wavelength present in the analysis [m]
- int: integer value [-]
- ang: angle of the line describing the Power Density Spectrum [-]
- A5m: Power Density at a wavelength of 5 m according to the line representing the Power Density Spectrum [m]

In this dissertation the value of $\lambda_{\text{max}}$ is 100 m, while the largest value of int equals 167, which means that the smallest wavelength present in the analysis is somewhat smaller than 0.6 m.

Apart from the properties of the initial longitudinal profile the model also requires some vehicle response properties; $f_{f_{\lambda,i}}$, $f_{r_{\lambda,i}}$, $\Delta f_{\lambda,i}$, and $\Delta r_{\lambda,i}$. The values of these response properties are determined by means of vehicle simulations. Within these simulations first of all a longitudinal profile is created which shows a pure sine of longitudinal unevenness with a wavelength which equals $\lambda_{\text{max}}$/int. Hereafter the required vehicle is simulated travelling over this single sine unevenness. The response of the vehicle is then analyzed and gives the four response properties required. It is clear that such an analysis has to be performed for all wavelengths in the analysis, i.e. 167 times.

The vehicle response properties, i.e. the values of $f_{\lambda,i}$, $r_{\lambda,i}$, $\Delta f_{\lambda,i}$ and $\Delta r_{\lambda,i}$, as determined for the 167 wavelengths for a certain vehicle/speed combination are stored in the computer. In this way a following analysis in which the particular vehicle/speed combination is a part of traffic does no longer require the vehicle simulations needed to determine the values of $f_{\lambda,i}$, $r_{\lambda,i}$, $\Delta f_{\lambda,i}$ and $\Delta r_{\lambda,i}$ for this particular vehicle/speed combination.

The model also needs the pavement rutting behaviour, i.e. $a_{p}$, $b_{p}$, $c_{p}$, $d_{p}$, $AL_{st}$ (= 2x$L_{st}$), $m_{l}$, $\sigma_{c_{st}}$, $m_{c}$ and RP. These values can be determined on the basis of the rutting performance model discussed in chapter 8.
9.5 COMPARISON OF THE REPEATED VEHICLE SIMULATION WITH THE MODEL

9.5.1 Introduction

In the previous sections it is explained that repeated vehicle simulation can be used to determine the development of vehicle induced longitudinal unevenness. Then the theory of a model explaining the development of this type of unevenness was discussed.

In this section some realistic examples of vehicle induced longitudinal unevenness are discussed. During the discussion the results of the repeated vehicle simulation are compared with the results of the model explaining the development of traffic induced longitudinal unevenness.

9.5.2 Pavement properties

Both the repeated vehicle simulation as well as the model need pavement properties considering initial longitudinal unevenness and rut depth development.

The initial longitudinal profile is determined on the basis of a power density spectrum represented by a straight line on a log-log scale, see equations 9.44 and 9.45. The value of the power density at a wavelength of 5 m is assumed to be $600 \times 10^9$ [m]. The angle of the straight line representing the power density spectrum of the new pavement is taken as $0.75$ [\].

The pavement considered is again a pavement with only a 1000 mm Zaanweg sand sub-base on a 60 MPa sub-grade. The rutting behaviour is determined for the situation that the pavement is only trafficked by 100 kN axle loads with a wheel load contact pressure of 700 kPa. These axle loads show a standard deviation of lateral wander of 200 mm, so that the pavement rutting parameters become equal to those given in section 9.3.2.

These parameters lead to a pavement that shows 15 mm relative rut depth after almost 140,000 standard axle load repetitions.

9.5.3 Results

The concrete block pavement is trafficked by the DAF-truck that has a 100 kN static rear axle load and a 69.9 kN static front axle load. The speed of this truck is 55 km/h. Three of these vehicles are considered. The first vehicle is a new DAF-truck. This truck has properly inflated tires ($k_f = 1,5000,000$ N/m and $k_r = 4,000,000$ N/m) and has new shock absorbers...
(c_1 = c_r = 30.000 Ns/m). The new truck is referred to as "vehicle 100 kN, standard".

The second truck considered here has overinflated tires to conceal heavy payloads, as a result of the overinflated tires the tire stiffnesses are doubled (k_u = 3,000,000 N/m and k_w = 8,000,000 N/m). The shock absorbers of this truck are new (c_f = c_r = 30.000 Ns/m). This truck is referred to as "vehicle 100 kN, 2xk".

The third truck considered here is an old truck which has worn shock absorbers (c_f = c_r = 10.000 Ns/m). The tires of this old truck are inflated properly (k_u = 1,5000,000 N/m and k_w = 4,000,000 N/m). This truck is referred to as "vehicle 100 kN, c/3".

9.5.3.1 Repeated vehicle simulation

For the repeated vehicle simulation an actual initial longitudinal profile is required as input. It is arbitrarily decided that the distance between two profile heights equals 0.2 m, so that the longitudinal profile that is used as input shows the same refinement as the measured longitudinal profiles, see chapter 11. This longitudinal profile is created on the basis of the earlier described parameters of the power density spectrum. It is assumed that the profile can be described by a Fourier series (42) that contains 171 different wavelengths. The maximum wavelength is (512 x 0.2 m) 102.4 m, so that the shortest wavelength considered becomes almost 0.6 m. The amplitudes of these 171 waves of unevenness are chosen such that they completely comply with the required Power Density Spectrum.

The Power Density Spectrum does not give any information about the phases of the waves of unevenness. To combine the 171 waves to one longitudinal profile the phases of the various waves are determined by means of a randomizer. The randomizer produces a random figure that varies from nil to one. For each wavelength the randomizer produced a different figure used to determine the phase of the wave of unevenness with that particular wavelength.

Of course the longitudinal profile of the considered pavement will change under traffic. To get insight into these changes Fourier analyses are performed on the various longitudinal profiles that develop during trafficking.

The results of the Fourier analyses are used to determine the ratio between the amplitude of longitudinal unevenness at a certain N_u and the amplitude of unevenness with the same wavelength in the initial longitudinal profile. When these ratios are plotted against the wavelength a relative amplitude spectrum is obtained. In figure 9.17 the development of the relative
amplitude spectrum for the pavement trafficked by "vehicle 100 kN, standard" only is given.

\[ A_{u'_{\lambda}}(N_{st}) / A_{u'_{\lambda}}(0) \]

\[ N_{st} \text{ [\ldots]} \]

\[ 0.1 \quad 1 \quad 10 \quad 100 \quad \lambda \text{ [m]} \]

**fig 9.17** Development of the relative amplitude spectrum for "vehicle 100 kN, standard".

Figure 9.17 shows that unevenness with a wavelength larger than about 10.7 m is amplified by the dynamical effects of vehicles. A peak develops at a wavelength of about 12.9 m. When \( N_{st} \) equals 138,800, and the repeated vehicle simulation thus is completed, a peak value of almost 2.5 is found, implying that the amplitude of longitudinal unevenness with a wavelength of about 12.9 m became 2.5 times larger than was the case in the initial longitudinal profile.

The figure also shows that unevenness with a wavelength shorter than about 2.5 m is smoothed by the dynamical effects of "vehicle 100 kN, standard". A minimum is found at a wavelength of about 1.5 m. At this wavelength the amplitudes of unevenness after 136,088 simulated vehicle passages are reduced to about 0.17 times the amplitudes in the initial longitudinal profile.

In figure 9.18 a similar plot as in figure 9.17 is given for the case that the pavement is trafficked by "vehicles 100 kN, 2xk". As is shown by this figure unevenness with a wavelength of about 1.5 m is now strongly reinforced by traffic. At \( N_{st} = 138,800 \) the amplitudes of unevenness at this wavelength are about 17 times larger than in the initial longitudinal profile.
"Vehicle 100 kN, 2xk" also amplifies longitudinal unevenness with wavelengths larger than about 10.2 m. A maximum of about 2.2 is found at a wavelength of about 12.9 m. Longitudinal unevenness with a wavelength between 3 m and 10.2 m is slightly smoothed by "vehicle 100 kN, 2xk" (figure 9.18).

According to figure 9.18 unevenness with a wavelength shorter than 1.3 m is at first strongly reduced by "vehicle 100 kN, 2xk". The lines plotted for $N_{st}$ equals 69,400 and $N_{st}$ equals 138,800 however show that a sudden amplification in this wavelength area occurs. A peak value of about 6.9 is found at a wavelength of about 0.75 m after 136,088 simulated vehicles, when $N_{st}$ equals 138,800. It is expected that this peak is a result of end effects that may occur since the Fourier analysis is performed on the actual longitudinal profiles, without the use of any tapering window (50).

Figure 9.19 gives the development of the relative amplitude spectrum for the case that the pavement is trafficked by "vehicle 100 kN, c/3". The figure shows that traffic now has a really disastrous effect on the development of longitudinal unevenness. After 6,804 simulated vehicles ($N_{st} = 6,940$) already a maximum $A_{ul}[N_{st}]/A_{ul}[0]$-value of 15.6 is found for a wavelength of about 1.85 m.

The huge effect of the vehicle on the development of longitudinal unevenness explains why the repeated simulation could not be completed up to
136,088 vehicle simulations ($N_{st} = 138,800$). After 6,804 simulated vehicles the simulated vehicle started to lose contact with the pavement surface. The vehicle now wants to jump. A linear vehicle model can not handle this situation and will result in negative axle loads, so that the simulation was aborted.

![Graph](image)

**fig 9.19** Development of the relative amplitude spectrum for "vehicle 100 kN, c/3".

Similar to the figures 9.17 and 9.18, figure 9.19 also shows that unevenness with a wavelength of about 12.5 m is amplified by "vehicle 100 kN, c/3". At this wavelength, when $N_{st}$ equals 6,940, the amplitude of unevenness is about 2.4 times larger than in the initial profile. Unevenness with a wavelength shorter than about 1.6 m is smoothed by "vehicle 100 kN, c/3". Again the peak at a wavelength shorter than 1 m is considered to be a result of end effects that develop when longitudinal unevenness is strongly affected by vehicle effects. As shown by figure 9.19 longitudinal unevenness with a wavelength between 3.5 m and 10.7 m is slightly smoothed by "vehicle 100 kN, c/3".

In the previous discussion it was shown that traffic affects longitudinal unevenness. Depending on the wavelength of unevenness traffic might either amplify or smoothen the amplitudes of longitudinal unevenness. It is clear that the changes in the longitudinal profile will affect the reactions of any vehicle to this longitudinal profile. As a result the standard deviation of the axle loads will vary with $N_{st}$. In figure 9.20 this is shown for the three vehicles considered here.
As shown by figure 9.20 "vehicle 100 kN, standard" affects the longitudinal profile in such a way that this vehicle will show a minor increase of the standard deviation of its axle loads in $N_{st}$. A much stronger increase is found when "vehicle 100 kN, 2xk" is considered. The strongest increase in the standard deviation of the axle loads however is found for "vehicle 100 kN, c/3". The effects of this vehicle on longitudinal unevenness are such that the standard deviation of the axle loads increase very rapidly with $N_{st}$.

In order to express the roughness of a pavement in one figure use can be made of the International Roughness Index, IRI (38, 43). Given a longitudinal profile this index is determined on the basis of the simulation of a two mass spring system or quarter car. While simulating the standardized IRI quarter car travelling at a speed of 80 km/h the movements in its shock absorber are summed up and divided by the length over which the simulation is completed. The IRI thus has no dimension but is mostly expressed in mm shock absorber movements per meter travelled length [mm/m] in order to come to convenient figures.

Of course the reactions of the IRI quarter vehicle to a longitudinal profile will depend on the longitudinal unevenness in this profile. The IRI-value of a longitudinal profile is thus effected by the influence of traffic on longitudinal unevenness. In figure 9.21 these effects are shown for the pavement discussed here, trafficked by the three types of vehicles.

The figure shows that the changes in the longitudinal profile that are induced by "vehicle 100 kN, standard" are such that a minor decrease of the IRI will develop over $N_{st}$. The other two vehicles, "vehicle 100 kN, 2xk" and "vehicle 100 kN, c/3"
"vehicle 100 kN, c/3" lead to an increasing IRI. Especially when the pavement is trafficked by "vehicle 100 kN, c/3" a very strong increase in IRI, i.e. a very strong decrease of the driving comfort, is shown.

This of course is in agreement with the previously discussed results which also showed that "vehicle 100 kN, c/3" has the strongest effects on the development of traffic induced longitudinal unevenness.

fig 9.21 Development of the IRI for the concrete block pavement trafficked by the three vehicles.

In this research, traffic induced longitudinal unevenness completely depends on the development of ruts in a pavement. As a result of the axle loads varying over the chainage the rut depth will also vary with the chainage. This implies that the rut depth as it develops during the repeated vehicle simulation will show a certain standard deviation. Based on this standard deviation and the average rut depth the rut depth with a 30% probability of exceeding can be determined. The development of this characteristic rut depth for the pavement trafficked by the three vehicles is given in figure 9.22.

In figure 9.22 also the development of the average or mean rut depth is given. The figure also gives the rut development that is found on the basis of the static axle loads. All rut depths plotted in the figure refer to the rut depth underneath a 1.2 m straight edge, i.e. the relative rut depth.

Figure 9.22 clearly shows that the dynamic interaction between pavement and vehicles results in a larger rut depth than would be found on the basis of the static axle loads. This implies that dynamical effects reduce the design life of a pavement in two ways. The first way is by introducing longitudinal unevenness that might reduce the driving comfort and as a result may cause premature failure as a result of roughness.
A good example of this kind of failure is given by the pavement trafficked only by "vehicle 100 kN, c/3". As discussed earlier the effects of this vehicle on the development of roughness are so strong that even before the average relative rut depth becomes 15 mm the vehicles will lose contact with the pavement. Both the development of the standard deviation of the axle loads, figure 9.20, as well as the development of the IRI, figure 9.21, show that the roughness in the pavement is strongly amplified by this vehicle. Premature failure as a result of roughness will in this case develop.

The second way in which dynamical effects reduce the design life of a pavement is that the rut depth that develops under dynamic axle loads is larger than the rut depth that develops under static axle loads. In figure 9.22 it is clearly shown that both the development of the average relative rut depth and the relative rut depth with a 30% probability of exceeding are affected by dynamical effects. It is shown that the rut depth that develops after a certain amount of traffic increases when the responses of the vehicles in traffic to longitudinal unevenness become more intensive. This of course reduces the design life based on rut development.

9.5.3.2 Roughness model

As discussed earlier a roughness model explaining the interaction between traffic and pavement has been developed. This model is much faster in determining what will happen than the repeated vehicle simulation. In this section the results of this model will be presented. The analyses performed
with the model equal the analyses performed by means of repeated vehicle simulation, section 9.5.3.1. The only small difference is that the maximum wavelength in the repeated vehicle simulation is 102.4 m while the maximum wavelength considered in the roughness model is 100 m.

The roughness model does not consider an actual longitudinal profile but concerns a set of 167 wavelengths and amplitudes of unevenness. Per wavelength the model explains how the initial amplitude of unevenness will develop with \( N_{st} \). Since the model considers a set of wavelengths and corresponding amplitudes of unevenness the creation of a relative amplitude spectrum of unevenness does not require any Fourier analysis.

The figures 9.23, 9.24 and 9.25 give the relative amplitude spectra of longitudinal unevenness as they are determined by the roughness model. These figures should thus show a large resemblance with the corresponding figures 9.17, 9.18 and 9.19 respectively.

![Graph](image)

**fig 9.23** Development of the relative amplitude spectrum for "vehicle 100 kN, standard."

Comparing figure 9.23 with figure 9.17 it is found that there are some differences between the model and the repeated vehicle simulation. The model shows that "vehicle 100 kN, standard" amplifies unevenness with wavelengths longer than about 10 m. A maximum magnification is found at a wavelength of about 12.4 m. After 138,800 equivalent standard axle load repetitions the initial amplitudes of unevenness with this wavelength are enlarged by a factor 2.2. These values closely resemble the values found on the basis of repeated vehicle simulation.

A larger difference between model and repeated vehicle simulation is found for wavelengths between 1.8 m and 3.5 m. The model shows that
longitudinal unevenness within this wavelength area is magnified. A maximum of about 1.65 is found at a wavelength of about 2.1 m. The results of the repeated vehicle simulation hardly showed the development of this magnification, see figure 9.17.

![Graph](image)

**Fig 9.24 Development of the relative amplitude spectrum for "vehicle 100 kN, 2xk".**

The situation when the pavement is trafficked by "vehicle 100 kN, 2xk" is much better. Now the model and the repeated vehicle simulation come to quite similar results, see figures 9.18 and 9.24. Figure 9.24 shows that unevenness with a wavelength of almost 1.5 m is strongly amplified by traffic. After 138,800 equivalent standard axle load repetitions the model shows that unevenness with this wavelength is amplified by a factor 14.5. A far smaller amplification of longitudinal unevenness is found for wavelengths between 9.7 m and 100 m. Within this wavelength area a maximum of about 2 is found at a wavelength of 12.8 m.

These values closely resemble the values found by the repeated vehicle simulation. What is even more, the shape of the amplifications of unevenness with wavelengths around 12.8 m closely resemble each other, as can be seen by comparing the figures 9.18 and 9.24.

Since the model does not require any Fourier analysis for the creation of a relative amplitude spectrum the results of the model do not show end effect peaks at small wavelengths.

Figure 9.25 shows that when the pavement is trafficked by "vehicle 100 kN, c/3" unevenness with a wavelength longer than about 10.4 m is magnified by traffic according to the model. The shape of these magnifications closely resembles the shape plotted in figure 9.19.
A much stronger amplification is found by the model at wavelength between 1.6 m and 3.9 m. Within this wavelength area a maximum amplification of about 10.3 is found at a wavelength of 1.7 m after only 6,940 standard axle load repetitions. Similar to the repeated vehicle simulation the model thus comes to the conclusion that the effects of "vehicle 100 kN, c/3" on the development of longitudinal unevenness are huge. The model however shows a smaller amplification than the repeated vehicle simulation which showed a maximum $\frac{Au_1[N_\text{st}]}{Au_1[0]}$-value of about 15.6 at $N_\text{st}$ equals 6,940. Again the model does not show any end effects in its relative amplitude spectrum.

Based on the comparison of the figures 9.23, 9.24 and 9.25 with the figures 9.17, 9.18 and 9.19 it can be concluded that the model explains the development of the relative amplitude spectrum as it develops under traffic quite well. The main difference between the model and the repeated vehicle simulation is that the maximum $\frac{Au_1[N_\text{st}]}{Au_1[0]}$-values in the relative amplitude spectra determined on the basis of repeated vehicle simulation are larger than in the spectra determined by the model. The trends in the relative amplitude spectra determined by the model however closely resemble the trends found by repeated vehicle simulation.

**Standard deviation of the axle loads**

The model considers a set of wavelengths and corresponding amplitudes of unevenness. This means that a simple multiplication will translate this...
information to the amplitudes of the dynamic components of the axle loads, see equation 9.26. The standard deviation caused by any number of complete cycles of a sine equals:

\[ S = \frac{1}{2} \sqrt{2} \sqrt{A^2} \quad (9.46) \]

The standard deviation of a signal that exists of "imax" sines which all fit any number of complete cycles in the signal (as is the case in a Fourier series) becomes:

\[ S = \frac{1}{2} \sqrt{2} \sqrt{\sum_{i=1}^{imax} A_i^2} \quad (9.47) \]

Where:
- \( i \): sine number [-]
- \( A_i \): amplitude of sine "i" [depending on signal dimension]
- \( S \): standard deviation [depending on signal dimension]
- \( imax \): number of sines that are considered [-]

Given equation 9.47 it becomes clear that the amplitudes of the dynamic components of the axle loads as determined by the model can easily be used to determine the standard deviation of the axle loads. In figure 9.26 the results of such calculations are given. The figure shows the development of the standard deviation of the axle loads of the three vehicles in relation to the number of equivalent standard axle loads that the static axle loads introduced by repeated trafficking of the pavement.

Like figure 9.20, figure 9.26 shows that the effects of "vehicle 100 kN, standard" on the development of longitudinal unevenness are such that only a minor increase in the standard deviation of the axle loads will occur.

Figure 9.26 shows that "vehicle 100 kN, c/3" results in an enormous increase of the standard deviation of the axle loads. The effects of "vehicle 100 kN, 2xk" are much stronger than the effects of "vehicle 100 kN, standard" but by far not so strong as the effects of "vehicle 100 kN, c/3".

Considering figure 9.20 this implies that the model comes to results that closely resemble the results of the repeated vehicle simulation. When the shape of the development of the standard deviation of the axle loads of "vehicle 100 kN, 2xk" is considered a difference between model and repeated vehicle simulation is however found.
Standard deviation of the rut depth

Like the repeated vehicle simulation the model also explains the development of longitudinal unevenness by considering a rut depth varying with the chainage. In the model the rut depth variations are seen as sines with different wavelengths and amplitudes. The standard deviation of the rut depth variations can thus easily be determined if the amplitudes of the various rut depth variations are known, see equation 9.46 and 9.47.

To determine the amplitudes in the rut depth variations a minor additional computation has to be performed. As shown in figure 9.13 the sine of the dynamic axle load may show a phase difference with the sine of unevenness that is causing the dynamic response. As a result the rut depth induced by the dynamic axle load may also show a phase difference with the sine of longitudinal unevenness that causes the dynamic response. As a result the phase of the sine of longitudinal unevenness before an axle load passage may differ from the phase of the sine of unevenness after the axle load passage.

The phase differences between longitudinal unevenness and the rut depth variations introduced by a vehicle passage imply that longitudinal unevenness will slowly move along the longitudinal profile, see figures 9.14, 9.15 and 9.16 and equations 9.34 to 9.43. The phase difference that is introduced by a single passage of the fictitious vehicle, in which the properties of all vehicle/speed combinations in traffic are taken into account, is given in equation 9.48.
\[ \Delta f_{r\lambda}[N_{st1} + Neq] = \Delta f_{r\lambda}[N_{st1}] + \frac{\lambda}{2\pi} \arctan \left( \frac{S_{fr\lambda}[N_{st1}, N_{st2}]}{1 + C_{fr\lambda}[N_{st1}, N_{st2}]} \right) \] (9.48)

where:
\( \Delta f_{r\lambda}[N_{st1}] \): phase difference between the sine of longitudinal unevenness at \( N_{st} = N_{u1} \) and the sine of longitudinal unevenness in the initial profile [m]

Similar to the explanation given in section 9.4.1.2 the value of \( \Delta f_{r\lambda}[N_{u2}] \) can now easily be determined on the basis of \( \Delta f_{r\lambda}[N_{u1}] \).

\[ \Delta f_{r\lambda}[N_{u2}] = \Delta f_{r\lambda}[N_{u1}] + \frac{N_{u2} - N_{u1}}{Neq} \times \frac{\lambda}{2\pi} \arctan \left( \frac{S_{fr\lambda}[N_{u1}, N_{u2}]}{1 + C_{fr\lambda}[N_{u1}, N_{u2}]} \right) \] (9.49)

Given the phase difference between longitudinal unevenness in the initial longitudinal profile and longitudinal unevenness with the same wavelength at some moment in \( N_{st} \), the amplitude of the sine in the rut depth can now be determined.

\[ Arda_{\lambda}[N_{u1}] = \left( \frac{Au_{\lambda}[N_{u1}].\sin^2(\Delta f_{r\lambda}[N_{u1}] \frac{2\pi}{\lambda}) + (Au_{\lambda}[N_{u1}].\cos(\Delta f_{r\lambda}[N_{u1}] \frac{2\pi}{\lambda}) - Au_{\lambda}[0])^2}{\sqrt{Au_{\lambda}[N_{u1}].\sin^2(\Delta f_{r\lambda}[N_{u1}] \frac{2\pi}{\lambda}) + (Au_{\lambda}[N_{u1}].\cos(\Delta f_{r\lambda}[N_{u1}] \frac{2\pi}{\lambda}) - Au_{\lambda}[0])^2}} \] (9.50)

where:
\( Arda_{\lambda}[N_{u1}] \): amplitude of the sine of the absolute rut depth with a wavelength \( \lambda \) at \( N_{u1} \) [mm]

It might be clear that the amplitude of the sine of the relative rut depth "\( Ardr_{\lambda}[N_{u1}] \)" equals \( Arda_{\lambda}[N_{u1}] \) divided by \( R_p \). On the basis of these amplitudes equation 9.46 can be used to determine the standard deviation of the relative rut depth. On the basis of a standard normal distribution, the relative rut depth with a 50% probability of exceeding can now be determined.

In figure 9.27 the development of the relative rut depth with a 30% probability of exceeding is given for the three types of traffic. In the figure also the development of the rut depth under static loadings is given. When this figure is compared to figure 9.22 a few things catch the eye.

First of all the development of the average relative rut depth, as determined by the model, is the same for all computations and equals the development of the rut depth under static loadings. This is a direct result of the model explaining the development of longitudinal unevenness on the basis
of linearized pavement rutting behaviour caused by the static loads.

The repeated vehicle simulation however showed that dynamic vehicle reactions not only cause rut depth variations, but also affect the development of the average rut depth. This secondary effect is thus not accounted for in the model.

Due to the increase of the axle load variations in $N_{ax}$ the repeated vehicle simulation shows a rut depth development that at larger $N_{ax}$ values might become more or less linear with $N_{ax}$. This is especially the case when the effects of traffic on the development of longitudinal unevenness are strong. In the model this behaviour is not observed. Again this is a result of the model explaining the development of rut depth variations on the basis of the linearized pavement rutting behaviour around the behaviour caused by the static loads.

![Graph showing relative rut depth vs. $N_{ax}$](image)

*Fig 9.27 Development of the relative rut depth with a 30% probability of exceeding.*

### 9.6 PAVEMENTS TRAFFICKED BY VARIOUS VEHICLE/SPEED COMBINATIONS

The model explaining the development of traffic induced longitudinal unevenness can easily be applied to get insight into the effects of traffic consisting of numerous vehicles with different properties and speeds. To give an idea of the results that are now obtained an example is discussed here in which traffic consists of well maintained vehicles ("vehicle 100 kN, standard") and vehicles with worn out shock absorbers ("vehicle 100 kN, c/3"). It is again assumed that all vehicles travel at 55 km/h. The parameters
describing the rutting behaviour of the pavement are equal to the parameters presented in paragraph 9.4.2. It is again assumed that all wheel loads are applied to the pavement by a 700 kPa contact pressure.

In figure 9.28 the relative amplitude spectrum of longitudinal unevenness as it is present after 138,800 standard axle load repetitions is presented. In this figure the effects of eight types of traffic are shown. Within these types of traffic the percentage of vehicles with worn shock absorbers varies from 0% to 70% with a step of 10%.

**Fig 9.28** Relative amplitude spectrum after 138,800 standard axle load repetitions for various types of traffic.

**Fig 9.29** Maximum amplification of longitudinal unevenness for wavelengths from 0.6 m to 5 m.
Especially the development of longitudinal unevenness with wavelengths around 2 m is strongly effected by the type of traffic that is using the pavement. As the percentage of vehicles with worn shock absorbers increases the traffic induced longitudinal unevenness in this wavelength area increases strongly. This is even stronger shown in figure 9.29 in which the maximum Au[138,800]/Au[0]-value for wavelengths between 0.6 and 5 m is plotted.

The traffic considered here also amplifies longitudinal unevenness with a wavelength of about 12.6 m. As is shown by figure 9.30 the effects of traffic on the amplification in this wavelength area is much smaller than the effects of traffic on unevenness with a wavelength of about 2 m. In figure 9.30 the maximum Au[138,800]/Au[0]-value for wavelengths between 5 m and 100 m, as it develops with growing N, is plotted.

![Diagram showing the relationship between magazine value and percentage of vehicle](image)

*fig 9.30  Maximum amplification of longitudinal unevenness for wavelengths from 5 m to 100 m.*

In figure 9.31 the development of the maximum Au[138,800]/Au[0]-value for both wavelength areas is plotted against the percentage of vehicles with worn shock absorbers. The figure clearly shows that the quality of the shock absorbers has a large influence on the development on longitudinal unevenness with wavelengths between 0.6 m and 5 m.
**Figure 9.31** Maximum amplification of longitudinal unevenness for wavelengths between 0.6 m and 5 m and for wavelengths between 5 m and 100 m.

The development of the standard deviation of the axle loads of the vehicles in traffic depends on the development of longitudinal unevenness that on its turn depends on the type of traffic that is using a pavement. In figure 9.32 the development of the standard deviation of the rear axle load of "vehicle 100 kN, standard" is plotted against $N_{st}$ for the various types of traffic. For the development of the standard deviation of the front axle load of "vehicle 100 kN, standard" figure 9.33 is given.

**Figure 9.32** Development of the standard deviation of the dynamic rear axle load of "vehicle 100 kN, standard" for different types of traffic.
Both the figures 9.32 and 9.33 show that the development of the standard deviation of the axle loads of "vehicle 100 kN, standard" strongly depends on the amount of vehicles with worn shock absorbers in traffic.

When for instance 30% of traffic has worn shock absorbers the standard deviation of the rear axle load of "vehicle 100 kN, standard" after 20,000 equivalent standard axle load repetitions equals the standard deviation that, in case that all vehicles have new shock absorbers, would only be reached after 138,800 equivalent standard axle load repetitions.

![Graph showing development of standard deviation of rear axle load](image)

**fig 9.33** Development of the standard deviation of the front axle load of "vehicle 100 kN, standard" for different types of traffic.

Considering the standard deviation of the front axle load the situation is somewhat better. For this axle load the standard deviation found after 30,000 equivalent standard axle load repetitions in case 30% of the vehicles have worn shock absorbers is about equal to the standard deviation that, in case all vehicles have new shock absorbers, would be found after 138,800 equivalent standard axle load repetitions.

As explained earlier the amplitudes of axle load variations are needed in order to determine the development of the standard deviations of the axle loads. Of course these amplitudes can also be plotted against the wavelength. In figure 9.34 this is done for the development of the amplitudes of the rear axle load variations of "vehicle 100 kN, standard" for the case that traffic consists of vehicles with new shock absorbers only.

Figure 9.34 shows that the reactions of the vehicle to longitudinal unevenness with wavelength shorter than about 1.9 m are strongly reduced during trafficking of the pavement. For the wavelengths longer than 1.9 m the
figure either shows an increase or a marginal decrease in the vehicle reactions.

![Graph](image)

**Figure 9.34** Development of the amplitudes of the rear axle load variations of "vehicle 100 kN, standard".

![Graph](image)

**Figure 9.35** Amplitudes of the rear axle load variations of "vehicle 100 kN, standard" at \( N_{st} = 138,800 \) for various types of traffic.

In figure 9.35 the amplitudes of the response of the rear axle of "vehicle 100 kN, standard" at \( N_{st} = 138,800 \) are presented for the different types of traffic. This figure shows that especially the development of vehicle responses in the wavelength area of 0.6 m to 5 m strongly depends on the type of traffic.

This is very clearly shown by figure 9.36. In this figure the maximum
amplitudes of the rear axle load responses for the wavelength area of 0.6 m to 5 m and the wavelength area of 5 m to 100 m at $N_{v} = 138,800$ are plotted against the percentage of vehicles with worn shock absorbers. This plot shows how strong the development of vehicle reactions in the wavelength area of 0.6 m to 5 m depends on the type of traffic. The vehicle response in the wavelength area of 5 m to 100 m also depends on the type of traffic that is using the pavement. As is shown by figure 9.36 the dependence of these vehicle responses on the type of traffic is however much smaller than the dependence of the responses with wavelengths of 0.6 m to 5 m.

![Graph showing development of maximum amplitudes of rear axle load variations.]

**Fig 9.36** Development of the maximum amplitudes of the rear axle load variations of "vehicle 100 kN, standard" for two wavelength areas at $N_{v} = 138,800$.

### 9.7 DISCUSSION

In this chapter the development of traffic induced longitudinal unevenness is discussed. It was shown that insight into the development of this type of longitudinal unevenness can quite easily be obtained by means of repeated vehicle simulation. Within such a repeated vehicle simulation each vehicle that will travel over the pavement has to be simulated. As a result the simple repeated vehicle simulation requires hours, days or even several days of computation time to simulate what will happen in a pavement during its design life.

In order to very strongly reduce the time needed for the determination of the development of longitudinal unevenness due to repeated vehicle passages a model has been developed. This model does not require repeated vehicle
simulation and, as a result, gives insight into the development of traffic induced longitudinal unevenness within a few minutes. It might be clear that this of course is an important advantage of the model when compared with the repeated vehicle simulation.

Another advantage of the model is that it is very flexible. The model is capable in determining the effects of traffic consisting of various vehicle/speed combinations, which of course means that the model is capable in considering more realistic types of traffic than the more rigid repeated vehicle simulation.

The model is based upon the assumption that pavement behaviour under dynamic loadings can be approached by considering the linearized pavement behaviour around the static loads in traffic. By comparing the outcome of the model with the outcome of the repeated vehicle simulation it was shown that this approach results in an underestimation of the effects of traffic on the development of longitudinal unevenness.

This underestimation is only limited when a pavement is loaded by vehicles that show "normal" dynamic reactions to longitudinal unevenness. When vehicles however show excessive reactions to longitudinal unevenness the average rut depth introduced by the dynamic wheel loads will be larger than the rut depth that follows from the static loads. As a result the outcome of the model, in such cases, does not completely equal the results of the repeated vehicle simulation.

Despite this difference between the model and the simulation in the case of excessive vehicle reactions, the model describes the phenomena that occur in a pavement trafficked by vehicles that show dynamic reactions very well. It gives insight into the wavelength areas in which longitudinal unevenness is amplified or smoothed by traffic. It furthermore gives insight in how strong the amplifications will be and gives a good impression of the effects of these amplifications on the development of the standard deviation of the axle loads and the rut depth. It is thus concluded that the model is a very powerful tool for explaining the effects of dynamic traffic reactions on the development of permanent surface deformation in concrete block pavements.

9.8 CONCLUSIONS

Based on the calculations discussed in this chapter a few conclusions about the effects of traffic on the development of both longitudinal and transversal unevenness can be made.

First of all it is concluded that the dynamic reaction of traffic will always reduce the rut depth based design life of a pavement. The reactions of
traffic will always cause a rut depth varying with the chainage. As a result the rut depth that develops will always show a certain standard deviation. This standard deviation of course increases the relative rut depth with a 30% probability of exceeding, which in the Netherlands is the design criterion for transversal unevenness. It is thus concluded that a proper design of a concrete block pavement should always include an estimation of the effects of dynamic traffic reactions on the development of transversal unevenness.

Of course the development of a rut depth varying with the chainage will have its effects on longitudinal unevenness in the wheel tracks. It is shown by both the model and the repeated vehicle simulation that these effects can either amplify or smoothen initial longitudinal unevenness. The wavelength area in which amplification or smoothing of unevenness occurs of course depends on the properties of the vehicles in traffic and their travelling speed. Considering the discussed DAF-truck with a 100 kN static rear axle load travelling at 55 km/h it was shown that unevenness with a wavelength up to about 1.5 m is strongly smoothed by traffic. Longitudinal unevenness with a wavelength of about 2 m is amplified by this vehicle/speed combination, as is longitudinal unevenness with a wavelength between about 10 m to 100 m.

Especially the amplification of longitudinal unevenness with a wavelength of about 2 m strongly depend on those properties of the vehicle that can easily change during the vehicle’s life. As a vehicle becomes older the quality of its shock absorbers will decrease. If the fleet of vehicles using a road network is not maintained this may lead to the situation that traffic for a large part consists of vehicles with worn shock absorbers. It was shown that this situation will have a devastating effect on the development of traffic induced longitudinal unevenness and rut depth variations. Depending on the percentage of vehicles with worn shock absorbers, the pavement considered in the previous sections might under these circumstances fail as a result of excessive longitudinal unevenness.

The calculations also show that increasing the stiffness of the tires, by increasing the tire pressure, has a very negative effect on the development of longitudinal unevenness. The negative effects of high tire stiffnesses are however not so strong as the effects of worn shock absorbers.

The calculations discussed clearly show that there is an interaction between pavement behaviour and the properties of traffic. A high quality pavement can very well be ruined by low quality, poorly maintained vehicles. Such traffic will introduce high dynamic axle loadings for which the pavement was not designed. Amplification of longitudinal unevenness in certain wavelength areas will in this case result in even higher axle loadings and very
rapidly damage the pavement to failure as a result of excessive rutting or longitudinal unevenness.
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**Practical applications**

10.1 **INTRODUCTION**

In the previous chapters the behaviour of concrete block pavements is discussed and analyzed. A rutting performance model explaining permanent surface deformation was developed on the basis of theoretical analyses of numerous concrete block pavements loaded by various traffic loadings. Then a model explaining the development of traffic induced longitudinal unevenness was derived.

In this chapter the importance of both the regression rutting model and the roughness model are shown by discussing a few examples.

10.2 **TRAFFIC**

In reality traffic consists of various types of vehicles that can travel at various speeds. In order to approach this situation, up to a certain extent, the dynamical properties of several vehicles are needed for input.

To get the properties of more vehicles the DAF-truck manufacturer was asked to provide the properties of some recent DAF-trucks. DAF provided the following properties of six of their trucks chassis including driver cabin: $M_{cw}$, $M_{tr}$, $k_{sf}$, $k_{sr}$, $c_{sf}$, $c_{sr}$, $k_{ft}$, $k_{fr}$, $l_{f}$ and $l_{r}$. Furthermore DAF gave numerous dimensions of their chassis.

Various body workshops mount a desired loading facility onto the DAF chassis and by doing so they turn the DAF into a practical tailor-made truck. Given the known dimensions of the chassis, a theoretical loading platform was mounted onto the chassis in this research to enable the computation of $I_{c}$, $I_{r}$, $J_{cw}$ and $M_{cw}$ of the unloaded DAF-trucks.

To get insight into the properties of typical Dutch public transport busses the ZWN bus-company was asked to give the dynamical properties of a modern bus used for public transport. They could only give a limited amount of information and provided the total weight of an empty modern bus and the distribution of this weight over the front and rear axle. Furthermore the total length of the bus was given, as well as the positions of the front and rear
axle. Also the number of seats and the maximum allowed number of standing passengers was given. From this information the properties of an empty bus were derived assuming that the bus is equipped with DAF axles: $M_{cw}$, $M_{rf}$, $M_{lf}$, $k_{rf}$, $k_{rf}$, $c_{sf}$, $c_{sr}$, $k_{sf}$, $k_{sr}$, $l_{f}$ and $l_{r}$. The value of $J_{cw}$ was estimated on the basis of the dimensions of the bus and the known load distribution.

In practice traffic largely consists of passenger cars. In order to be able to take also into account the effects of passenger cars, the properties of a passenger car are retrieved from the properties of such a car modelled by a complex full vehicle model developed by TNO.

It is remarked here that this TNO model also contains a linear shock absorber in the tires of the vehicles. The half vehicle model used in this research is not equipped with a damper representing the shock absorbing properties of tires.

Combined with the properties of the DAF-truck presented in chapter 9, the properties of 4 trucks are now determined, see table 10.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>truck 1, empty</th>
<th>truck 2, empty</th>
<th>truck 3, empty</th>
<th>truck 4, empty</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{cw}$ [kg]</td>
<td>5,500 $^a$</td>
<td>5,806 $^b$</td>
<td>6,158 $^b$</td>
<td>7,001 $^b$</td>
</tr>
<tr>
<td>$J_{cw}$ [kg m²]</td>
<td>9,640 $^a$</td>
<td>34,765.9 $^c$</td>
<td>48,263.4 $^c$</td>
<td>83,424.7 $^c$</td>
</tr>
<tr>
<td>$M_{rf}$ [kg]</td>
<td>800 $^a$</td>
<td>748 $^d$</td>
<td>748 $^d$</td>
<td>748 $^d$</td>
</tr>
<tr>
<td>$M_{lf}$ [kg]</td>
<td>1,300 $^a$</td>
<td>1,414 $^d$</td>
<td>1,414 $^d$</td>
<td>1,414 $^d$</td>
</tr>
<tr>
<td>$k_{sf}$ [N/m]</td>
<td>430,000 $^a$</td>
<td>598,000 $^d$</td>
<td>598,000 $^d$</td>
<td>598,000 $^d$</td>
</tr>
<tr>
<td>$k_{sr}$ [N/m]</td>
<td>700,000 $^a$</td>
<td>604,000 $^d$</td>
<td>604,000 $^d$</td>
<td>604,000 $^d$</td>
</tr>
<tr>
<td>$c_{sf}$ [Ns/m]</td>
<td>30,000 $^a$</td>
<td>22,000 $^d$</td>
<td>22,000 $^d$</td>
<td>22,000 $^d$</td>
</tr>
<tr>
<td>$c_{sr}$ [Ns/m]</td>
<td>30,000 $^a$</td>
<td>30,200 $^d$</td>
<td>30,200 $^d$</td>
<td>30,200 $^d$</td>
</tr>
<tr>
<td>$k_{if}$ [N/m]</td>
<td>1,500,000 $^a$</td>
<td>2,400,000 $^d$</td>
<td>2,400,000 $^d$</td>
<td>2,400,000 $^d$</td>
</tr>
<tr>
<td>$k_{ir}$ [N/m]</td>
<td>4,000,000 $^a$</td>
<td>4,800,000 $^d$</td>
<td>4,800,000 $^d$</td>
<td>4,800,000 $^d$</td>
</tr>
<tr>
<td>$l_{f}$ [m]</td>
<td>1.07 $^a$</td>
<td>1.52 $^b$</td>
<td>1.91 $^b$</td>
<td>2.72 $^b$</td>
</tr>
<tr>
<td>$l_{r}$ [m]</td>
<td>2.18 $^a$</td>
<td>3.38 $^b$</td>
<td>3.79 $^b$</td>
<td>4.17 $^b$</td>
</tr>
<tr>
<td>$L_{st,f}$ [kN]</td>
<td>44.559 $^a$</td>
<td>46.579 $^b$</td>
<td>47.456 $^b$</td>
<td>48.795 $^b$</td>
</tr>
<tr>
<td>$L_{st,r}$ [kN]</td>
<td>29.921 $^a$</td>
<td>31.507 $^b$</td>
<td>34.079 $^b$</td>
<td>41.000 $^b$</td>
</tr>
</tbody>
</table>

**table 10.1 Properties of the unloaded trucks.**

In table 10.1, and the tables that follow, the sources of the various figures are indicated by letters. These letters refer to the following sources:

- a: These properties are explicitly given by Sweere (39),

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b: These figures are explicitly given by DAF for the chassis without any loading facility. The effects of the mounted load carrying facility on the figures given by DAF are estimated.

c: The moment of inertia of the unloaded trucks is estimated on the basis of the known weight of the unloaded trucks, the known weight distribution over the front and rear axle and the known dimensions of the truck.

d: These figures are explicitly given by DAF.

e: Figures that follow directly from information explicitly given by the ZWN bus-company under the assumption that the bus is equipped with DAF axles.

f: The moment of inertia of the empty bus is estimated on the basis of the known weight of the body work, its known dimensions and the load distribution over the front and rear axles.

g: Figures that are retrieved from a true vehicle model developed by TNO.

h: Figures estimated on the basis of the figures marked with the letter "g". They are chosen such that a somewhat smaller passenger car with a total weight of 1000 kg is simulated.

In the tables 10.1 to 10.6 some figures are not accompanied by any letter. These figures refer to the values of $M_{cw}$, $J_{cw}$, $l_r$ and $l_l$ of loaded vehicles and follow directly from the properties of unloaded vehicles provided that the load is precisely known.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Bus, empty</th>
<th>Bus, 32 passengers</th>
<th>Bus, 83 passengers</th>
<th>passenger car 1</th>
<th>passenger car 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{cw}$ [kg]</td>
<td>8,768 $^e$</td>
<td>11,008 $^d$</td>
<td>14,578 $^d$</td>
<td>1,400 $^g$</td>
<td>909 $^h$</td>
</tr>
<tr>
<td>$J_{cw}$ [kg]</td>
<td>140,529 $^f$</td>
<td>158,368 $^d$</td>
<td>185,756 $^d$</td>
<td>1,500 $^g$</td>
<td>750 $^h$</td>
</tr>
<tr>
<td>$M_{lf}$ [kg]</td>
<td>748 $^d$</td>
<td>748 $^d$</td>
<td>748 $^d$</td>
<td>70 $^g$</td>
<td>45 $^h$</td>
</tr>
<tr>
<td>$M_r$ [kg]</td>
<td>1,414 $^d$</td>
<td>1,414 $^d$</td>
<td>1,414 $^d$</td>
<td>70 $^g$</td>
<td>45 $^h$</td>
</tr>
<tr>
<td>$k_{sf}$ [N/m]</td>
<td>598,000 $^d$</td>
<td>598,000 $^d$</td>
<td>598,000 $^d$</td>
<td>40,000 $^g$</td>
<td>26,000 $^h$</td>
</tr>
<tr>
<td>$k_{sr}$ [N/m]</td>
<td>604,000 $^d$</td>
<td>604,000 $^d$</td>
<td>604,000 $^d$</td>
<td>35,000 $^g$</td>
<td>22,500 $^h$</td>
</tr>
<tr>
<td>$c_{sf}$ [Ns/m]</td>
<td>22,000 $^d$</td>
<td>22,000 $^d$</td>
<td>22,000 $^d$</td>
<td>3,000 $^g$</td>
<td>1,900 $^h$</td>
</tr>
<tr>
<td>$c_{sr}$ [Ns/m]</td>
<td>30,200 $^d$</td>
<td>30,200 $^d$</td>
<td>30,200 $^d$</td>
<td>4,000 $^g$</td>
<td>2,600 $^h$</td>
</tr>
<tr>
<td>$k_{lf}$ [N/m]</td>
<td>2,400,000 $^d$</td>
<td>2,400,000 $^d$</td>
<td>2,400,000 $^d$</td>
<td>400,000 $^g$</td>
<td>300,000 $^h$</td>
</tr>
<tr>
<td>$k_{lr}$ [N/m]</td>
<td>4,800,000 $^d$</td>
<td>4,800,000 $^d$</td>
<td>4,800,000 $^d$</td>
<td>400,000 $^g$</td>
<td>300,000 $^h$</td>
</tr>
<tr>
<td>$l_f$ [m]</td>
<td>3.62 $^e$</td>
<td>3.83</td>
<td>3.99</td>
<td>1.50 $^g$</td>
<td>1.30 $^h$</td>
</tr>
<tr>
<td>$l_r$ [m]</td>
<td>2.13 $^e$</td>
<td>1.92</td>
<td>1.76</td>
<td>1.25 $^g$</td>
<td>1.10 $^h$</td>
</tr>
<tr>
<td>$L_{sf,t}$ [kN]</td>
<td>39.161 $^c$</td>
<td>43.352</td>
<td>51.059</td>
<td>6.92 $^g$</td>
<td>4.524 $^h$</td>
</tr>
<tr>
<td>$L_{sr,t}$ [kN]</td>
<td>67.953 $^c$</td>
<td>85.713</td>
<td>112.993</td>
<td>8.17 $^g$</td>
<td>5.266 $^h$</td>
</tr>
</tbody>
</table>

**table 10.2 Properties of buses and luxury cars.**

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**Practical applications**

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For the busses this is the case by assuming that one person has a weight of 70 kg. The moment of inertia of person when seated is estimated to be 14.12 kg.m\(^2\) while a standing person has a moment of inertia of 17.09 kg.m\(^2\). According to the ZWN bus-company the bus has 45 seats and space for 38 standing passengers.

The loading of the trucks was based upon a payload with a specific gravity of 1000 kg/m\(^3\). The load is equally spread over the loading area of the truck. Each truck is loaded such that the static rear axle load \( L_{\text{sa},r} \) becomes 80 kN, 100 kN, 120 kN, 140 kN or 160 kN. The tables 10.3 to 10.6 give those vehicle properties that are affected by the payload.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>truck 1, 80 kN</th>
<th>truck 1, 100 kN</th>
<th>truck 1, 120 kN</th>
<th>truck 1, 140 kN</th>
<th>truck 1, 160 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{cw}} ) [kg]</td>
<td>12,458.5</td>
<td>15,237.5</td>
<td>18,016.5</td>
<td>20,759.0</td>
<td>23,574.4</td>
</tr>
<tr>
<td>( J_{\text{cw}} ) [kg m(^2)]</td>
<td>23,525.24</td>
<td>28,371.2</td>
<td>33,662.1</td>
<td>39,690.5</td>
<td>46,704.7</td>
</tr>
<tr>
<td>( l_1 ) [m]</td>
<td>1.763</td>
<td>1.870</td>
<td>1.944</td>
<td>1.998</td>
<td>2.040</td>
</tr>
<tr>
<td>( l_2 ) [m]</td>
<td>1.437</td>
<td>1.330</td>
<td>1.256</td>
<td>1.202</td>
<td>1.160</td>
</tr>
<tr>
<td>( L_{\text{sa},r} ) [kN]</td>
<td>62.67</td>
<td>69.90</td>
<td>77.14</td>
<td>84.26</td>
<td>91.59</td>
</tr>
<tr>
<td>( L_{\text{sa},r} ) [kN]</td>
<td>80.01</td>
<td>100.00</td>
<td>120.00</td>
<td>139.76</td>
<td>160.02</td>
</tr>
</tbody>
</table>

*Table 10.3  Properties of truck 1 for 5 different payloads.*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>truck 2, 80 kN</th>
<th>truck 2, 100 kN</th>
<th>truck 2, 120 kN</th>
<th>truck 2, 140 kN</th>
<th>truck 2, 160 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{cw}} ) [kg]</td>
<td>11,967.7</td>
<td>14,509.0</td>
<td>17,050.3</td>
<td>19,591.6</td>
<td>22,132.9</td>
</tr>
<tr>
<td>( J_{\text{cw}} ) [kg m(^2)]</td>
<td>71,818.34</td>
<td>83,088.7</td>
<td>93,743.5</td>
<td>104,159.7</td>
<td>114,560.7</td>
</tr>
<tr>
<td>( l_1 ) [m]</td>
<td>2.763</td>
<td>2.969</td>
<td>3.113</td>
<td>3.219</td>
<td>3.301</td>
</tr>
<tr>
<td>( l_2 ) [m]</td>
<td>2.137</td>
<td>1.931</td>
<td>1.787</td>
<td>1.681</td>
<td>1.599</td>
</tr>
<tr>
<td>( L_{\text{sa},r} ) [kN]</td>
<td>58.48</td>
<td>63.36</td>
<td>68.27</td>
<td>73.20</td>
<td>78.11</td>
</tr>
<tr>
<td>( L_{\text{sa},r} ) [kN]</td>
<td>79.99</td>
<td>100.01</td>
<td>120.01</td>
<td>139.99</td>
<td>159.98</td>
</tr>
</tbody>
</table>

*Table 10.4  Properties of truck 2 for 5 different payloads.*
<table>
<thead>
<tr>
<th>Parameter</th>
<th>truck 3, 80 kN</th>
<th>truck 3, 100 kN</th>
<th>truck 3, 120 kN</th>
<th>truck 3, 140 kN</th>
<th>truck 3, 160 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{cw}$ [kg]</td>
<td>12,015.2</td>
<td>14,566.3</td>
<td>17,117.3</td>
<td>19,668.3</td>
<td>22,219.3</td>
</tr>
<tr>
<td>$J_{cw}$ [kg m²]</td>
<td>95,978.7</td>
<td>111,657.1</td>
<td>126,345.0</td>
<td>140,523.1</td>
<td>154,465.2</td>
</tr>
<tr>
<td>$l_r$ [m]</td>
<td>3.202</td>
<td>3.440</td>
<td>3.607</td>
<td>3.730</td>
<td>3.826</td>
</tr>
<tr>
<td>$l_t$ [m]</td>
<td>2.498</td>
<td>2.260</td>
<td>2.093</td>
<td>1.970</td>
<td>1.874</td>
</tr>
<tr>
<td>$L_{st,r}$ [kN]</td>
<td>58.93</td>
<td>63.93</td>
<td>68.93</td>
<td>73.95</td>
<td>78.92</td>
</tr>
<tr>
<td>$L_{st,t}$ [kN]</td>
<td>80.00</td>
<td>100.00</td>
<td>120.01</td>
<td>139.99</td>
<td>160.02</td>
</tr>
</tbody>
</table>

*Table 10.5 Properties of truck 3 for 5 different payloads.*

<table>
<thead>
<tr>
<th>Parameter</th>
<th>truck 4, 80 kN</th>
<th>truck 4, 100 kN</th>
<th>truck 4, 120 kN</th>
<th>truck 4, 140 kN</th>
<th>truck 4, 160 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{cw}$ [kg]</td>
<td>11,801.2</td>
<td>14,263.0</td>
<td>16,724.9</td>
<td>19,186.7</td>
<td>21,648.5</td>
</tr>
<tr>
<td>$J_{cw}$ [kg m²]</td>
<td>146,291.3</td>
<td>171,966.7</td>
<td>195,838.6</td>
<td>218,650.6</td>
<td>240,815.0</td>
</tr>
<tr>
<td>$l_r$ [m]</td>
<td>3.946</td>
<td>4.252</td>
<td>4.468</td>
<td>4.629</td>
<td>4.753</td>
</tr>
<tr>
<td>$l_t$ [m]</td>
<td>2.954</td>
<td>2.648</td>
<td>2.432</td>
<td>2.271</td>
<td>2.147</td>
</tr>
<tr>
<td>$L_{st,r}$ [kN]</td>
<td>56.84</td>
<td>60.97</td>
<td>65.10</td>
<td>69.22</td>
<td>73.34</td>
</tr>
<tr>
<td>$L_{st,t}$ [kN]</td>
<td>80.00</td>
<td>99.99</td>
<td>119.99</td>
<td>140.00</td>
<td>160.00</td>
</tr>
</tbody>
</table>

*Table 10.6 Properties of truck 4 for 5 different payloads.*

It is assumed that the properties of enough vehicles are now known in order to come to a realistic representation of real traffic. In the following section some examples of road deterioration caused by traffic consisting of various vehicles will be discussed. Within these sections use is made of the vehicle properties presented in this section.

### 10.3 BUS-ROUTE OVER RESIDENTIAL STREET

#### 10.3.1 Introduction

For this example a typical Dutch residential street is considered. In the Netherlands concrete block pavements in such streets are mostly constructed without a proper structural design. Before the houses in the residential area can be constructed, construction traffic has to get access to the building area. Given the poor subgrades in the Netherlands, this often means that first of all the bearing capacity of the subsoil in the total building area has to be improved. In the Netherlands most of the times the residential area is covered...
with a sand layer for this reason before the construction works can begin.

When the construction works in the residential area reaches its completion the sand layer automatically becomes the sub-structure of the residential streets. This is mostly done without any further structural considerations.

In this example such a residential street is considered. The sand sub-base of this street has a thickness of 800 mm. The subgrade below the sub-base consists of a sandy-clay as can be found in larger areas in The Netherlands and has a stiffness of 60 MPa.

In the example it is assumed that during the first six years after completion the residential street provides access for residential cars and an occasional service truck only. Hereafter a bus-route is detoured in such a way that the residential street becomes part of it. The public bus-company now has the advantage of a complete new residential area with new costumers.

10.3.2 Traffic and pavement behaviour

For this example it is assumed that the pavement, during the first six years after construction, is trafficked by (150 passenger cars/day x 7 days/week x 52 weeks/year =) 54,600 passenger cars per year per direction. Half of these passenger cars have the properties of "passenger car 1" while the other half have the properties of "passenger car 2".

Apart from the passenger cars the pavement is also trafficked by three trucks per week. These trucks represent garbage collectors, movers, furniture deliverers, garden requirement deliverers (soil, sand, tiles) and other occasional trucks using the residential street. One of the trucks has the properties of "truck 1, 80 kN", another truck is represented by "truck 2, 80 kN" while the last truck is formed by "truck 3, 100 kN". Given the assumption that one of each truck will use the pavement each week, each truck passes 52 times per year.

The traffic described above is applied to the pavement for a period of six years. This type of traffic is referred to as "traffic 1".

After six years a bus-route is detoured over the considered pavement. It is assumed that this will result in (50 busses/day x 6 days/week + 26 busses/day x 1 day/week) x 52 weeks/year =) 16,952 busses/year/direction. In total 2,236 empty busses "bus, empty" will pass per year. By far the most busses are fairly full and carry 32 passengers, which implies that about 71% of the seats in the bus are occupied. It is assumed that 10,348 of these busses represented by "bus, 32 passengers" pass through the street per year per
direction. The remaining 4,368 bus passages per year consider "bus, 83 passengers" and thus refer to completely full busses. By adding these bus movements to "traffic 1", "traffic 2" is obtained.

Based on the properties of traffic (static axle loads and their distribution) the rutting behaviour of the pavement under consideration can be determined. Hereto it is assumed that all traffic will show a standard deviation of lateral wander of 200 mm and that all loads are applied through a contact pressure of 707 kPa. It is accepted that by doing so, the minimal effects of the passenger cars are overestimated. The pavement rutting properties presented in table 10.7 are found on the basis of the rutting performance model discussed in chapter 8. It is assumed that the sand in the substructure has the properties of an average Dutch sand, i.e. Zaanweg sand.

<table>
<thead>
<tr>
<th>Rutting parameter</th>
<th>Traffic 1</th>
<th>Traffic 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_p$ [mm]</td>
<td>5.541</td>
<td>5.933</td>
</tr>
<tr>
<td>$b_p$ [-]</td>
<td>0.223</td>
<td>0.266</td>
</tr>
<tr>
<td>$\Lambda_{cst}$ [kN]</td>
<td>97.36</td>
<td>109.04</td>
</tr>
<tr>
<td>$m_i$ [-]</td>
<td>9.942</td>
<td>7.982</td>
</tr>
<tr>
<td>$\sigma_{cst}$ [kPa]</td>
<td>707</td>
<td>707</td>
</tr>
<tr>
<td>$m_c$ [-]</td>
<td>3.648</td>
<td>2.957</td>
</tr>
<tr>
<td>RP [-]</td>
<td>1.540</td>
<td>1.492</td>
</tr>
</tbody>
</table>

*Table 10.7 Rutting properties of the pavement in the residential street under the two types of traffic ($c_p$ and $d_p$ equal 0).*

10.3.3 Analysis of the pavement behaviour under traffic

10.3.3.1 Introduction

As explained in chapter 9 the model describing the development of traffic induced longitudinal unevenness is based upon the development of rut depth variations as a result of dynamic axle loads. The model considers the effects of various wavelengths ($\lambda = 100/i$ m for $i = 1$ to 167) individually.

In this section the results of such an analysis are presented and discussed. The initial longitudinal profile was assumed to show a straight line Power Density Spectrum on a log-log scale. This Power Density Spectrum is described by a Power Density at a wavelength of 5 m "$a_5m$" of $600 \times 10^9$ [m$^2$/m] and a slope of 0.75 on a log-log scale. With these properties the initial longitudinal profile equals the longitudinal profile of a new concrete block.
pavement as measured in Zaanstad, see Allanstraat in table 11.3.
For this analysis it was assumed that all vehicles in traffic will travel over the pavement with a speed of 55 km/h.

10.3.3.2 Rut development

As explained in chapter 9 the roughness model gives insight into the development of ruts and the variation in the rut depth over the chainage. In figure 10.1 the results of the analysis are presented.

The figure clearly shows the effects of the introduction of the busses on the pavement after 6 years. As shown by the figure the accumulation of rut depth strongly increases when the busses are introduced. The figure also shows the effects of the dynamic axle loads on the rut development. The relative rut depth with a 30% probability of exceeding, upper line in figure 10.1, is clearly larger than the average relative rut depth (middle continuous line).

The rut depth with a 30% probability of exceeding is determined on the basis of the rut depth variations, with various wavelengths, that are a result of dynamic axle load variations. These rut depth variations are represented by sines, from the amplitudes of these sines the standard deviation in the rut depth can be computed. Based on a standard normal distribution the rut depth with a 30% probability of exceeding can be computed.

**fig 10.1** Rut development on the residential street.

Applying a 15 mm relative rut depth with a 30% probability of exceeding as a failure criterion, the figure shows that the pavement will fail
after about 9.5 years. This implies that a good, six year old residential street, that hardly shows any damage (relative rut depth with a 30% probability of exceeding of about 5 mm), is brought to failure due to rutting within a four year period by the introduction of the public transport busses.

Given the difference between the average relative rut depth after a certain number of load repetitions and the rut depth with a 30% probability of exceeding after the same number of load repetitions, it will be clear that the pavement will also suffer under the introduction of longitudinal unevenness, as is discussed in the next section.

10.3.3.3 Development of longitudinal unevenness

Rut depth variations always result in changes in longitudinal unevenness in the wheeltracks. Depending on the wavelength, the amplitudes of unevenness can either be amplified or reduced by the effects of traffic. In figure 10.2 the development of the maximum amplification of unevenness, the maximum value of $\bar{A}_{u}[N]/\bar{A}_{u}[0]$, is presented.

![Development of the maximum $\bar{A}_{u}[N]/\bar{A}_{u}[0]$-value on the residential street.](image)

Similar to figure 10.1, figure 10.2 also shows that the residential street will hardly be damaged during the first six years of being in service. A maximum value of $\bar{A}_{u}[N]/\bar{A}_{u}[0]$ of 1.37 is found at the end of the six year period in which the pavement is trafficked by "traffic 1". Directly after the introduction of the busses "traffic 2" the maximum amplification of longitudinal unevenness strongly increases. This implies that the introduction
of "traffic 2" will not only result in a strong increase in rut depth, but also in the rapid development of longitudinal unevenness.

To get insight into the wavelengths at which "traffic 2" has such strong amplifying effects, figure 10.3 is given. In this figure the development of the relative amplitude spectrum is presented.

Figure 10.3 shows that both "traffic 1" and "traffic 2" have a smoothing effect on longitudinal unevenness with wavelength shorter than about 1.7 m. The reduction of the amplitudes of longitudinal unevenness with these shorter wavelengths is a process that is not changed after six years when the busses are introduced.

Figure 10.3 also indicates that longitudinal unevenness with wavelengths around 2 m are amplified by "traffic 1". This amplification is however only limited and results in a $A_{u_0}/[N]/A_{u_0}[0]$-value of 1.37 for a wavelength of 2 m at the end of the six year period in which "traffic 1" used the residential street.

As is shown by the figure the amplification of longitudinal unevenness with a wavelength of about 2 m becomes especially strong after the introduction of the busses, "traffic 2". One year after the introduction of the busses the $A_{u_0}/[N]/A_{u_0}[0]$-value for a wavelength of 2 m increased from 1.37 to 1.90. At the end of the almost 17 year period for which the analysis is performed this value has become 3.22. This shows that due to the introduction of the busses with "traffic 2" especially unevenness with a wavelength of about 2 m is strongly amplified.

![Diagram](image)

**fig 10.3** Development of $A_{u_0}/[N]/A_{u_0}[0]$ for all the considered wavelengths for the residential street.
As shown by figure 10.3, "traffic 1" reduces longitudinal unevenness within the wavelength area of about 4.2 m to about 11 m. The smoothing effects of "traffic 1" in this wavelength area however remain limited. Unevenness with a wavelength longer than 11 m is amplified by "traffic 1". Within this wavelength area a maximum $A_{u_1}[N]/A_{u_0}[O]$-value of about 1.16 at a wavelength of 14.3 m is found at the end of the six year period in which "traffic 1" trafficked the pavement.

By the introduction of "traffic 2" a different trend is observed in the wavelength area of 4.2 m to 100 m. As is shown by figure 10.3, "traffic 2" has a smoothing effect on longitudinal unevenness with a wavelength of about 4.2 m to about 14.3 m. As a result, a large part of the longitudinal unevenness with wavelengths between 11 m and 14.3 m, induced by "traffic 1", is quickly smoothed after the introduction of "traffic 2".

Due to the effects of "traffic 2" amplification of longitudinal unevenness with wavelength longer than about 14.3 m takes place. A maximum $A_{u_2}[N]/A_{u_0}[O]$-value of about 1.28 is found at a wavelength of about 16.7 m one year after the introduction of the busses with "traffic 2". At the end of the almost 17 year period this value becomes about 1.63.

10.3.3.4 Development of the standard deviation of axle loads

As discussed, traffic will affect the longitudinal profile of a concrete block pavement as a result of the introduction of a varying rut depth. Since the dynamic axle loads of a vehicle depend on the longitudinal unevenness of a pavement, the standard deviation of the axle loads will be affected by the introduction of a varying rut depth. In figure 10.4 this is shown by presenting the development of the standard deviation of the rear axle load of the vehicles in both "traffic 1" and "traffic 2" over an almost 17 year period. Similar to figure 10.4, figure 10.5 gives the development of the standard deviation of the dynamic front axle loads of the vehicles that travel over the residential street.

As is shown by figure 10.4 the standard deviations of the rear axle loads of the various vehicles in "traffic 1", directly after opening of the street to traffic, show a strong decrease. This is a result of the strong smoothing effect of "traffic 1" on longitudinal unevenness with a wavelength shorter than about 1.7 m. After this initial reduction, a five and a half year period is observed, in which the standard deviations of the rear axle loads remain more or less constant.

Directly after this period, "traffic 2" is introduced. As a result of the change of traffic now most vehicles will show an increasing standard deviation of the rear axle load. Some vehicles however first show a minor

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A decrease in the standard deviation of the rear axle load directly followed by an increase.

![Diagram showing the development of the standard deviation of the rear axle loads on the residential street.]

**Fig 10.4** Development of the standard deviation of the rear axle loads on the residential street.

Figure 10.5 gives the development of the standard deviations of the front axle loads of the vehicles in both "traffic 1" and "traffic 2". As is shown by this figure a much more complex plot is now obtained.

![Diagram showing the development of the standard deviation of the front axle loads on the residential street.]

**Fig 10.5** Development of the standard deviation of the front axle loads on the residential street.
It is again observed that there is a significant influence of traffic on the longitudinal unevenness and the standard deviation of the front axle loads during the first half year. Most vehicles trafficking the pavement during this first half year show a decrease of the standard deviation of the front axle load. The standard deviation of the front axle load of "truck 1, 80 kN" however shows a slight increase.

After the first half year period it is again observed that the standard deviations of the front axle loads remain more or less constant until "traffic 2" is introduced. Directly after the introduction of "traffic 2" strong changes in the standard deviations of the front axle loads develop. Four out of eight vehicles in "traffic 2" ("passenger car 1", "passenger car 2", "truck 1, 80 kN" and "Bus, 83 passengers") either show an immediate increase in the standard deviation of the front axle load or a slight decrease directly followed by a strong increase.

The other four vehicles in traffic ("truck 2, 80 kN", "truck 3, 100 kN", "bus, empty" and "bus 32 passengers") however show a strong decrease of the standard deviation of the front axle loads directly after the introduction of "traffic 2". This decrease lasts for about two years (depending on the vehicle) and then changes into a mild but steady increase.

To get more insight into the development of the standard deviation of the axle loads it is necessary to consider the amplitude spectra of the axle loads of the vehicles trafficking the residential street as is done in the next paragraph.

10.3.3.5 Amplitudes of the dynamic component of axle loads

The previously discussed standard deviation of the axle loads develops of course as a result of load variations due to the dynamic response of the vehicles to the longitudinal profile. Of course the responses of the vehicles depend on two things: the amount of longitudinal unevenness at different wavelengths and the intensity of the vehicle reactions to longitudinal unevenness with different wavelengths.

To get a better understanding of these vehicle reactions the amplitude spectra of the axle loads of the various vehicles, for three moments in time, are created, see appendix 10.1. Of these 16 plots 2 characteristic ones are discussed here. Both these amplitude spectra refer to "truck 2, 80 kN". In figure 10.6 three dynamic front axle load amplitude spectra of this particular vehicle are presented.

As shown by figure 10.6 the amplitude spectrum of the dynamic front axle load of "truck 2, 80 kN" after 6 years mainly differs from the initial
amplitude spectrum at wavelength shorter than about 4 m. As shown the
dynamic responses of the front axle with wavelength shorter than about 1.7 m
(about 9 Hz at 55 km/h) show a strong decrease over the first six years after
opening of the residential street to traffic.

For wavelengths between 1.7 m and about 4.2 m a minor increase in the
vehicle responses is observed over the first six years. At a wavelength of
about 1.9 m (about 8 Hz at 55 km/h) the dynamic front axle reactions show a
maximum after six years.

![Amplitude spectra of the front axle load of "truck 2, 80 kN" on the residential street at three moments in time.](image)

In the wavelength area of about 4.2 m to 100 m the responses of the
front axle of "truck 2, 80 kN" hardly change during the first six years of
traffic. In this wavelength area a slight decrease of the front axle
responses is observed for wavelengths between 4.2 m and about 11 m,
whereas a minor increase is observed for wavelengths longer than about 11
m. Within this wavelength area a peak value in front axle responses during
the first six years of trafficking is found at a wavelength of 10 m (about 1.5
Hz at 55 km/h).

An explanation for the observations discussed above can be found by
considering the development of the relative amplitude spectrum of longitudinal
unevenness, figure 10.3. In those wavelength areas in which this spectrum
shows a decrease in amplitudes during the first six years of trafficking, the
front axle responses of "truck 2, 80 kN" show a decrease too. For those
wavelength areas where an amplification of longitudinal unevenness due to
traffic is observed in figure 10.3, figure 10.6 shows an increase in the
amplitudes of the front axle responses of "truck 2, 80 kN".

The relative amplitude spectrum shows that the effects of traffic on longitudinal unevenness with a wavelength of 10 m remain very limited during the first six years. This explains why the front axle responses for wavelengths between 4.2 m and 100 m hardly change during the first six years. Within this wavelength area a maximum in front axle reactions is found at 10 m, the amplitudes of unevenness with this wavelength are hardly affected by "traffic 1" so that the front axle responses hardly change.

The development of the standard deviation of the front axle load of "truck 2, 80 kN" during the first six years of trafficking is now explained. Directly after construction this standard deviation shows a strong decrease as a result of the strong decrease in front axle responses with a wavelength shorter than about 1.7 m. After this immediate decrease a stabilisation in the standard deviation is observed. To explain this stabilisation one has to consider the following.

When traffic smoothens longitudinal unevenness with a certain wavelength, the vehicle responses at that particular wavelength will decrease. As a result the dynamic forces that cause the smoothing of longitudinal unevenness at this particular wavelength will show a decrease. When traffic however amplifies longitudinal unevenness the vehicle responses will increase during trafficking. As a result the dynamic forces that cause the amplification of longitudinal unevenness thus increase during trafficking. This implies that, during trafficking, the smoothing effects of traffic become less and less strong when compared to the amplifying effects of traffic, and vice versa.

When the standard deviation of an axle load shows a decrease directly after a road is opened for traffic, this decrease thus will always, at some moment in time, change in an increase.

The observed stabilisation after the initial decrease of the standard deviation of the front axle load of "truck 2, 80 kN" during the first six years is thus a result of a decreasing importance of smoothing effects and an increasing importance of the amplifying effects.

The principle discussed above explains the development of the standard deviation of the front axle of "truck 2, 80 kN" after the introductions of the busses. As observed in figure 10.5 this standard deviation now shows a decreasing value during the first two years after the introduction of "traffic 2". Hereafter a slightly increasing standard deviation of the front axle load is observed.

By considering figure 10.6 it is found that the initial decrease after the introduction of "traffic 2" is a result of the smoothing effects of traffic on longitudinal unevenness with wavelengths of about 10 m and the ongoing

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smoothing effects on longitudinal unevenness with wavelengths shorter than 1.7 m.

The increase observed after about 2 years is a result of the amplifying effects of "traffic 2" on longitudinal unevenness with wavelengths around 2 m (7.6 Hz at 55 km/h) and wavelengths around 16.7 m (1.1 Hz at 55 km/h). As a result of these amplifying effects of "traffic 2" the amplitude spectrum of the front axle load of "truck 2, 80 kN" develops peak values at the mentioned wavelengths, see figure 10.6.

The development of the standard deviation of the rear axle load of "truck 2, 80 kN" is quite similar to the development of the standard deviation of the front axle load of this vehicle, see figures 10.4 and 10.5. After the introduction of the busses with "traffic 2" the standard deviation of the rear axle load of "truck 2, 80 kN" however, contrary to the standard deviation of the front axle, shows an immediate increase.

\[ \text{fig 10.7 Amplitude spectra of the rear axle load of "truck 2, 80 kN" on the residential street at three moments in time.} \]

The development of the standard deviation of the rear axle load of "truck 2, 80 kN" after the introduction of the busses can again be explained by considering the development of the amplitude spectrum of the dynamic responses of the rear axle load, figure 10.7. As is shown by this figure a very strong peak value in the rear axle responses develops at a wavelength of about 2 m. Apparently the rear axle is very sensitive for longitudinal unevenness with a wavelength of about 2 m when the vehicle is travelling at a speed of 55 km/h. Considering the relative amplitude spectra presented in figure 10.3 it is found that "traffic 2" has strong amplifying effects on longitudinal unevenness.
with wavelengths around 2 m.

The fact that the standard deviation of the rear axle load of "truck 2, 80 kN" show an immediate increase after the introduction of the busses is now explained as follows.

"Traffic 2" has a strong amplifying effect on longitudinal unevenness with a wavelength of about 2 m. The sensitivity of the reactions of the rear axle of "truck 2, 80 kN" travelling at 55 km/h to longitudinal unevenness with this wavelength are quite strong. As a result the amplitude spectrum of the dynamic rear axle load of this vehicle shows a strong increase of the responses with wavelengths around 2 m. The increase of these dynamic axle load responses is so strong that their effect on the standard deviation of axle load responses is dominant. As a result the standard deviation of the rear axle load of "truck 2, 80 kN" shows an immediate increase after the introduction of the busses.

10.4 FACTORY GATE

10.4.1 Introduction

For this example a plant producing precast concrete products (such as paving blocks, edge restraints, sewer pipes and tiles) is considered. It is assumed that the plant is situated besides a canal, which is often the case in the Netherlands. The raw materials for producing the concrete products can as a result be delivered to the plant in bulk by ship. The concrete blocks and other road building materials are transported by trucks from the factory to the clients.

As a result there is a firm difference between incoming traffic at the gate (empty trucks) and outgoing traffic (loaded trucks). It is thus expected that the lane for outgoing traffic will deteriorate much faster than the lane for incoming traffic.

To get insight into the effects of this difference a full analysis is made of the pavement behaviour that will develop under these circumstances.

10.4.2 Traffic and pavement behaviour

For this example it is assumed that the gate of the plant is equipped with a small shelter for the gatekeeper. This gatekeeper shelter is situated in the middle of the gate so that the gatekeeper can hold records of both incoming and outgoing traffic. As a result of the traffic guiding effects of the gatekeeper shelter it is assumed that the amount of lateral wander is limited:
standard deviation of lateral wander $\sigma_{lw} = 150$ mm.

The traffic that leaves the factory consists of four types of trucks; the properties of these trucks have already been presented in table 10.1 and tables 10.3 to 10.6. Of course different payload magnitudes occur. It is assumed that during nine hours per day, 2 trucks leave the factory per hour. The factory is open for 5 days per week in 52 weeks per year, so that 4,680 trucks enter and leave the factory per year. The trucks that leave the factory are presented in table 10.8. Of course exactly the same trucks enter the factory empty to collect their payload. It is again assumed that all the vehicle tires have a contact pressure of 707 kPa.

It is further assumed that the pavement design for the factory gate was based upon achieving a 12 year design life for the heavy outgoing traffic. The design criterion was a 15 mm relative rut depth. During the design the static axle loadings were considered, no attention was paid to the development of longitudinal unevenness due to dynamic axle load variations. The lanes for incoming and outgoing traffic have the same concrete block pavement structure.

<table>
<thead>
<tr>
<th>Static rear axle load [kN]</th>
<th>&quot;truck 1&quot; [-/year]</th>
<th>&quot;truck 2&quot; [-/year]</th>
<th>&quot;truck 3&quot; [-/year]</th>
<th>&quot;truck 4&quot; [-/year]</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>390</td>
<td>390</td>
<td></td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>585</td>
<td>585</td>
<td>624</td>
<td>117</td>
</tr>
<tr>
<td>140</td>
<td>195</td>
<td>195</td>
<td>936</td>
<td>234</td>
</tr>
<tr>
<td>160</td>
<td></td>
<td></td>
<td>312</td>
<td>117</td>
</tr>
</tbody>
</table>

*Table 10.8 Traffic leaving the factory, in vehicles per year.*

For the substructure design use is made of the rutting performance models discussed in chapter 8. The total height of the substructure of the pavement for the factory gate is fixed and equals 900 mm, the subgrade has a 60 MPa modulus. Based on the properties of the outgoing traffic (static axle loads and their distribution) the rutting behaviour of the pavement under consideration was determined for several base layer thicknesses using a "M. Havelaarweg" type of base material in combination with a "Zaanweg" type of sand sub-base.

A 12 year 15 mm relative rut depth design life was obtained by applying a 300 mm thick "M. Havelaarweg" type base on a 550 mm thick "Zaanweg" type sand sub-base. The pavement has a 50 mm thick crusher sand bedding layer.

In table 10.9 the parameters describing the pavement rutting behaviour under both incoming and outgoing traffic are presented.
In table 10.9 the $c_p$ and $d_p$ values presented for the pavement under the incoming traffic are nil. Both parameters however have a very small value, that is rounded to nil.

For the following analysis of the pavement behaviour under dynamic axle loads (sections 10.4.3 and 10.4.4) it was assumed that the initial longitudinal unevenness equals the initial longitudinal unevenness as discussed for the "bus-route" example, see paragraph 10.3. The speeds of the trucks entering or leaving the factory yard is estimated to be 25 km/h.

<table>
<thead>
<tr>
<th>Rutting parameter</th>
<th>Outgoing traffic</th>
<th>Incoming traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_p$ [mm]</td>
<td>3.723</td>
<td>0.368</td>
</tr>
<tr>
<td>$b_p$ [-]</td>
<td>0.064</td>
<td>0.017</td>
</tr>
<tr>
<td>$c_p$ [mm]</td>
<td>0.693</td>
<td>0</td>
</tr>
<tr>
<td>$d_p$ [-]</td>
<td>0.136</td>
<td>0</td>
</tr>
<tr>
<td>$AL_{st}$ [kN]</td>
<td>150.22</td>
<td>46.62</td>
</tr>
<tr>
<td>$m_1$ [-]</td>
<td>13.468</td>
<td>10.659</td>
</tr>
<tr>
<td>$\sigma_{c,st}$ [kPa]</td>
<td>707</td>
<td>707</td>
</tr>
<tr>
<td>$m_c$ [-]</td>
<td>5.400</td>
<td>4.289</td>
</tr>
<tr>
<td>RP [-]</td>
<td>1.243</td>
<td>1.4545</td>
</tr>
</tbody>
</table>

*table 10.9 Rutting properties of the concrete block pavement for the gate of the plant under both incoming and outgoing traffic.*

**10.4.3 Analysis of the pavement behaviour under incoming traffic**

**10.4.3.1 Introduction**

In this section the behaviour of the pavement under the dynamic axle loads of incoming traffic is discussed. For this analysis use is made of the roughness model discussed in chapter 9.

As will be shown, the part of the pavement trafficked by the unloaded trucks does hardly show any deterioration at all. Even after a 12 year period traffic was not able to harm the pavement in any way.

The fact that traffic consisting of unloaded trucks does not really harm the concrete block pavement structure can be understood by considering the number of equivalent axle loads that are introduced to the pavement by both outgoing and incoming traffic. On the basis of the properties of the pavement
under incoming traffic \( (AL_{ul} = 46.62 \text{ kN}, m_t = 10.659) \) the static axle loads of incoming traffic result in 5,119 standard axle load repetitions per year, while the static axle loads of outgoing traffic would induce 459,552,422 standard axle load repetitions per year \( (AL_{ul} = 46.62 \text{ kN}) \). This implies that outgoing traffic represents about 89,780 times more standard axle load repetitions than incoming traffic (based on the behaviour of the pavement under incoming traffic).

It is clear that the damaging effects of the unloaded trucks are only a fraction of the damaging effects of the loaded trucks. Of course especially the outgoing trucks with a 160 kN rear axle load cause a lot of damage.

Since incoming traffic is not heavy enough to damage the pavement no real developments can be detected. In the following sections however some results are shown without detailed discussions.

### 10.4.3.2 Rut development

In figure 10.8 the relative rut depth development of the pavement under the dynamic axle loads of incoming traffic is presented. The figure clearly shows that the empty trucks are not capable in harming the pavement in any way. As shown by the figure a minor rut depth develops immediately after the factory gate is opened for traffic. Apart from this immediate rut development hardly any further rutting is observed.

![Rut development under incoming traffic on the factory gate over a 12 year period.](image)

**fig 10.8** Rut development under incoming traffic on the factory gate over a 12 year period.
10.4.3.3 Development of longitudinal unevenness

As is shown by figure 10.8 only a minor rut depth is introduced by incoming traffic. Since traffic induced longitudinal unevenness is caused by rut depth variations the effects of the incoming traffic on the development of longitudinal unevenness can only be limited too. This is shown by figure 10.9 in which the development of relative amplitude spectrum over a 12 year period is given.

![Graph showing development of longitudinal unevenness](image)

**Fig 10.9** Development of $A_{u_2}/A_{u_1}$ for all the considered wavelengths on the factory gate over a 12 year period.

10.4.3.4 Development of the standard deviation of axle loads

Since the longitudinal profile of the pavement is hardly effected by the influence of incoming traffic, the standard deviation of the axle loads remains more or less constant over the years. In figure 10.10 the development of this standard deviation is plotted for "truck 1, empty" and "truck 2, empty".

Figure 10.10 again clearly shows that incoming traffic can not harm the pavement that was designed on the basis of the static axle loadings of the loaded outgoing trucks. Hardly any ruts are introduced so that the rut depth variations remain limited. As a result, traffic can hardly affect the longitudinal profile, so that the standard deviation of the axle loads is almost constant over a 12 year period.
For the other vehicles in the incoming traffic similar plots are found. These vehicles thus also show that there is hardly any development in the reactions of the vehicles to the pavement over a 12 year period.

![Graph showing standard deviation of axle loads]

**fig 10.10** Standard deviation of the axle loads of "Truck 1, empty" and "Truck 2, empty" on the factory gate over a 12 year period.

### 10.4.3.5 Amplitudes of the dynamic component of axle loads

In the previous sections it was shown that incoming traffic can not harm the pavement in any way. As a result really no developments caused by traffic induced damage could be observed. The same holds for the responses of the vehicles to the longitudinal profile of the lane for incoming traffic. Since incoming traffic can not influence longitudinal unevenness, the responses of the various empty trucks remain nearly constant over a 12 year period.

Nevertheless an impression of these responses is given in figures 10.11 and 10.12.
fig 10.11  Amplitude spectra of the axle loads of "Truck 2, empty" on the factory gate over the 12 year period.

fig 10.12  Amplitude spectra of the axle loads of "Truck 3, empty" on the factory gate over the 12 year period.
10.4.4 Analysis of the pavement behaviour under outgoing traffic

10.4.4.1 Introduction

As explained in section 10.4.2 the factory gate was designed in such a way that the pavement has a 12 year 15 mm relative rut depth design life under the static axle loadings of outgoing traffic. This implies that, contrary to incoming traffic, outgoing traffic will damage the pavement and cause the development of ruts. As a result outgoing traffic will probably introduce rut depth variations and thus affect longitudinal unevenness.

In the following sections the behaviour of the concrete block pavement on the factory gate under the dynamic axle loadings of outgoing traffic is discussed. It is shown that the loaded trucks will heavily damage the pavement.

10.4.4.2 Rut development

In figure 10.13 the rut development results of the analysis are presented. The figure shows that the pavement design answers to the demand that a 12 year design life has to be achieved (see the development of the average relative rut depth). Due to the dynamic effects of traffic, rut depth variations are introduced.

These rut depth variations, with various wavelengths, are represented by sines. From the amplitudes of these sines the standard deviation in the rut depth can be computed. Based on a standard normal distribution the rut depth with a 30% probability of exceeding can be computed. Similar to this also the rut depth with a 70% probability of exceeding can be computed.

Figure 10.13 shows that the standard deviation of the rut depth becomes so large that a decrease of the rut depth with a 70% probability of exceeding is observed after about 10 years. This can not be the case and is a result of the difference between the distribution of the actual rut depth variations and the standard distribution used to compute the rut depth with a 70% probability of exceeding.

The rut development under outgoing traffic clearly shows that after a period of about 8 years serious problems occur. The pavement will now rapidly collect damage due to the dynamic effects of traffic and it is clear that the earlier mentioned design life of 12 years is not a realistic number. As indicated the 15 mm relative rut depth design life becomes about 10 years.
when the relative rut depth with a 30% probability of exceeding is used as the design criterion, as is the case in the Netherlands.

![Rut development under outgoing traffic on the factory gate over a 12 year period.](image)

**Fig 10.13** Rut development under outgoing traffic on the factory gate over a 12 year period.

### 10.4.4.3 Development of longitudinal unevenness

The rut depth variations that develop under traffic again result in changes in longitudinal unevenness. In figure 10.14 the development of the maximum amplification of longitudinal unevenness, the maximum value of $A_{u,\lambda}[N]/A_{u,\lambda}[0]$, under outgoing traffic is presented.

As is shown by figure 10.14, the maximum value of $A_{u,\lambda}[N]/A_{u,\lambda}[0]$ under outgoing traffic shows a disastrous development after about 8 years of trafficking. The development of the maximum value of $A_{u,\lambda}[N]/A_{u,\lambda}[0]$ during the first 8 years can however hardly be distinguished in figure 10.14. For that reason the development of the maximum value of $A_{u,\lambda}[N]/A_{u,\lambda}[0]$ during the first years is presented in figure 10.15.

The figures 10.14 and 10.15 only give insight into the development of the maximum value of $A_{u,\lambda}[N]/A_{u,\lambda}[0]$ under outgoing traffic. As explained earlier, traffic can either amplify or reduce the amount of longitudinal unevenness, depending on the wavelength. To get insight into the wavelengths at which the outgoing traffic has the strongest amplifying effects, figure 10.16
is given. In this figure the development of the relative amplitude spectrum is presented for the 12 year period.

**Fig 10.14** Development of the maximum $A_{u, f[N]} / A_{u, f[0]}$-value under outgoing traffic on the factory gate over a 12 year period.

**Fig 10.15** Development of the maximum $A_{u, f[N]} / A_{u, f[0]}$-value under outgoing traffic on the factory gate during the first 8 years.
Figure 10.16 strongly indicates that especially longitudinal unevenness with wavelengths of about 0.9 m and about 8 m are amplified by outgoing traffic. The amplification at these wavelengths after 12 years of trafficking is so strong that the development during the first 10 years can not be distinguished in figure 10.16. Therefore figure 10.17 is presented, which
gives the development of the relative amplitude spectrum during the first 10 years of trafficking.

This last figure again shows that serious problems will develop after about 8 years. As is shown by figure 10.17 the developments in the relative amplitude spectrum between 4 and 8 years are not so strong as the developments during the shorter period between 8 and 10 years.

10.4.4.4 Development of the standard deviation of axle loads

As was shown in the previous sections, outgoing traffic has strong effects on the longitudinal profile of the pavement of the factory gate. Since the axle load variations (dynamic axle loads) depend on the amount of longitudinal unevenness and the distribution thereof over the wavelengths, this implies that the standard deviation of axle loads will show a certain development too. In this section the development of the standard deviation of the axle loads of four trucks in the outgoing traffic are presented.

![Standard deviation of axle loads](image)

**fig 10.18** Development of the standard deviation of the axle loads of "truck 1, 120 kN" and "truck 2, 120 kN" on the factory gate over a 12 year period.

In figure 10.18 the development of the standard deviation of the axle loads of "truck 1, 120 kN" and "truck 2, 120 kN" are plotted. In figure 10.19 a similar plot is presented for the development of the standard deviation of the axle loads of "truck 3, 140 kN" and "truck 4, 140 kN".
Both the figures 10.18 and 10.19 show that problems start to develop after about 8 years of trafficking. Due to the accumulated longitudinal unevenness and the rutting behaviour of the pavement, the development of longitudinal unevenness will now rapidly cause deterioration of the pavement.

An increasing magnification of longitudinal unevenness with wavelengths of about 0.9 and about 8 m will develop. As a result the standard deviation of all axle loads shows an increasing growth too, leading to failure as a result of longitudinal unevenness.

![Development of the standard deviation of the axle loads of "truck 3, 120 kN" and "truck 4, 120 kN" on the factory gate over a 12 year period.](image)

**10.4.4.5 Amplitudes of the dynamic component of axle loads**

The standard deviation of the axle loads of course is formed by axle load variations due to the dynamic response of the vehicles to the longitudinal profile. To get a better understanding of these vehicle reactions the amplitude spectra of the axle loads of two of the outgoing vehicles are presented.

The first amplitude spectra presented refer to "truck 2, 120 kN". In figure 10.20 five front axle load spectra of this particular vehicle are presented. Due to the enormous amplitudes found after 12 years of trafficking the development of the axle load spectrum during the first 8 years can not be distinguished. For this reason the development of the amplitude spectrum of
the front axle of "truck 2, 120 kN" during the first 8 years is presented in figure 10.21.

fig 10.20 Amplitude spectra of the front axle load of "truck 2, 120 kN" on the factory gate at five moments in time.

fig 10.21 Amplitude spectra of the front axle load of "truck 2, 120 kN" on the factory gate at four moments in time.
Similar to the figures 10.20 and 10.21, the figures 10.22 and 10.23 give the development of the rear axle load spectrum of "truck 2, 120 kN". In these figures it is again found that enormous axle load variations develop at wavelengths of about 0.9 m and about 8 m.

**Fig 10.22** Amplitude spectra of the rear axle load of "truck 2, 120 kN" on the factory gate at five moments in time.

**Fig 10.23** Amplitude spectra of the rear axle load of "truck 2, 120 kN" on the factory gate at four moments in time.
The development of the amplitude spectra of the axle loads of "truck 2, 120 kN" can of course be understood by considering the figures 10.16 and 10.17 which show that especially longitudinal unevenness with wavelengths of about 0.9 m and about 8 m are strongly amplified by the outgoing traffic.

In the figures 10.24 and 10.25 the development of the amplitude spectra of the axle loads of "truck 3, 140 kN" over the first eight years of trafficking are presented.

![Amplitude spectra of the front axle load of "truck 3, 140 kN" on the factory gate at four moments in time.](image)

**fig 10.24** Amplitude spectra of the front axle load of "truck 3, 140 kN" on the factory gate at four moments in time.

By considering figure 10.25 it can be seen that the rear axle loads of especially the more heavily loaded trucks will cause the amplification of longitudinal unevenness with a wavelength of about 8 m. At this particular wavelength the amplitudes of the rear axle response of "truck 3, 140 kN" show an increase right from the first moment after construction.

The less heavily loaded trucks (for instance "truck 2, 120 kN", see figure 10.23) first show a decrease in the peak amplitude of the rear axle load at wavelengths shorter than 8 m. Only after some time of trafficking the peak value at wavelength of about 8 m will start to increase as a result of traffic induced longitudinal unevenness. This shows that the less heavily loaded trucks can not be held responsible for the amplification of longitudinal unevenness with a wavelength of about 8 m.
**10.5 CONCLUSIONS**

From the design examples, discussed in the previous section, it can be concluded that the rutting behaviour of a concrete block pavement is not solemnly determined by the subgrade modulus and the substructure design. The magnitude of the static wheel loads and the contact pressures also effect the rutting behaviour of a pavement. In other words the concrete block pavement rutting behaviour parameters: $a_p$, $b_p$, $c_p$, $d_p$, $AL_{st}$, $m_1$, $σ_{c\,st}$, $m_c$ and RP depend on both the substructure design (and subgrade modulus) and the static properties of traffic. Furthermore the amount of lateral wander also effects the rutting behaviour of a concrete block pavement.

It is thus concluded that it is not possible to assign a certain behaviour to a concrete block pavement when the properties of traffic are not known.

Furthermore the examples show that the dynamic characteristics of the vehicles that compose the overall traffic are also of great importance for the pavement rutting behaviour. These dynamic characteristics determine the responses of the vehicles in traffic to longitudinal unevenness. Depending on these responses (combined with the properties of the pavement under traffic) larger or smaller variations of the rut depth will develop over the length of the pavement. The more intense the vehicle responses, the larger the rut depth variations, the shorter the rut depth related pavement design life.
The last parameter that effects the development of rutting is the initial longitudinal unevenness, which was not varied in the examples. It is however clear that the magnitude of initial longitudinal unevenness determines the initial vehicle responses. The larger these initial responses the larger the rut depth variations that will develop as a result. The rut depth related design life will thus decrease with an increase of the initial pavement roughness.

Given the above it is thus concluded that concrete block pavements can not be designed properly on the basis of the static loads of traffic. As shown by both the bus-route example and the factory gate example the dynamic responses of vehicles to longitudinal unevenness will result in rut depth variations over the length of the pavement. Neglecting the dynamic responses of the vehicles in traffic implies that these rut depth variations are not considered and thus lead to an over-estimation of the design life that will be achieved in practice.

As shown by the examples the rut depth varying over the length of the pavement also effects longitudinal unevenness. It is shown that it is not possible to say that traffic smoothens or amplifies initial longitudinal unevenness. Depending on the wavelength of unevenness, traffic either amplifies or smoothens existing unevenness. It is however clear that both the amplifying and the smoothing effects at a particular wavelength depend on the amplitude of axle load variations with that same particular wavelength. As a result of smoothing the amplitudes of axle load variations will decrease, reducing the smoothing effects of traffic in time.

Similarly the amplitudes of the axle load variations at a wavelength where longitudinal unevenness is amplified become larger and larger due to the traffic induced longitudinal unevenness. As a result the amplifying effects of traffic on longitudinal unevenness become stronger in time.

This implies that the amplifying effects of traffic on longitudinal unevenness at a certain wavelength eventually become more important than the smoothing effects of that same traffic at another wavelength. Eventually each concrete block pavement will thus suffer from traffic induced longitudinal unevenness.

The initial longitudinal unevenness of a concrete block pavement is very important for the further development of longitudinal unevenness. The rougher the initial profile, the larger the rut depth variations over the length of the pavement, the larger the effects of rut depth on the longitudinal unevenness. Since the roughness amplifying effects of traffic will eventually become of larger importance than the roughness smoothing effects this implies that initial pavement roughness not only has a negative effect on the development of rut depth variations but also on the further development of longitudinal unevenness.
From the discussion presented in this chapter it can be concluded that the interaction between traffic and a concrete block pavement is a complex matter. This interaction depends on the combination of the properties of the concrete block pavement structure and the properties of traffic. The models developed in this research are powerful tools to analyze this interaction and thus enable the design of concrete block pavements on the basis of the pavement - traffic interaction. In this type of pavement design both the development of traffic induced transversal and longitudinal unevenness is considered, taking into account the dynamic responses of traffic to pavement roughness.
Appendix 10.1

Amplitude spectra of axle loads

**fig a10.1.1** Amplitude spectra of the front axle loads of "passenger car 1" travelling over the residential street at 55 km/h.

**fig a10.1.2** Amplitude spectra of the rear axle loads of "passenger car 1" travelling over the residential street at 55 km/h.
**fig a10.1.3** Amplitude spectra of the front axle loads of "passenger car 2" travelling over the residential street at 55 km/h.

**fig a10.1.4** Amplitude spectra of the rear axle loads of "passenger car 2" travelling over the residential street at 55 km/h.
fig a10.1.5 Amplitude spectra of the front axle loads of "truck 1, 80 kN" travelling over the residential street at 55 km/h.

fig a10.1.6 Amplitude spectra of the rear axle loads of "truck 1, 80 kN" travelling over the residential street at 55 km/h.
**Fig a10.1.7** Amplitude spectra of the front axle loads of "truck 2, 80 kN" travelling over the residential street at 55 km/h.

**Fig a10.1.8** Amplitude spectra of the rear axle loads of "truck 2, 80 kN" travelling over the residential street at 55 km/h.
Fig a10.1.9 Amplitude spectra of the front axle loads of "truck 3, 100 kN" travelling over the residential street at 55 km/h.

Fig a10.1.10 Amplitude spectra of the rear axle loads of "truck 3, 100 kN" travelling over the residential street at 55 km/h.
Amplitude spectra of the front axle loads of "bus, empty" travelling over the residential street at 55 km/h.

Amplitude spectra of the rear axle loads of "bus, empty" travelling over the residential street at 55 km/h.
**fig a10.1.13**  
Amplitude spectra of the front axle loads of "bus, 32 passengers" travelling over the residential street at 55 km/h.

**fig a10.1.14**  
Amplitude spectra of the rear axle loads of "bus, 32 passengers" travelling over the residential street at 55 km/h.
**fig a10.1.15**  
Amplitude spectra of the front axle loads of "bus, 83 passengers" travelling over the residential street at 55 km/h.

**fig a10.1.16**  
Amplitude spectra of the rear axle loads of "bus, 83 passengers" travelling over the residential street at 55 km/h.
11
Profile measurements and verification and validation of the roughness prediction model

11.1 INTRODUCTION

For this research longitudinal and transversal profile measurements were performed on the seven concrete block test pavements described in chapter 5. These measurements give the deformed surface of in-service concrete block pavements. The goal of this research is to describe the development of traffic induced surface deformation, therefore the outcome of the profile measurements can be used for verification purposes. The profile measurements discussed here have been done in July 1991 on the test pavements in Rotterdam and in August 1991 on the test pavements in Zaanstad.

11.2 PROFILE MEASUREMENTS

11.2.1 The Automatic Road ANalyzer

11.2.1.1 Introduction

The road profiles as discussed in this chapter are measured by means of theAutomatic Road Analyzer, ARAN, see figure 11.1. The ARAN consists of a vehicle, a Chevy-van, equipped with highly sophisticated measuring devices and an onboard computer. A more powerful micro-computer is used to process the data obtained by the ARAN. This micro-computer is situated at the office building of the Road and Hydraulic Engineering Division (DWW) of the Ministry of Traffic, Public Works and Water Management.

The ARAN has built-in sub-systems which together measure the longitudinal and transversal unevenness as well as the longitudinal and transversal slopes of a road. Furthermore the ARAN is equipped with three
video cameras so that videologging is possible. Together with the mentioned micro-computer these sub-systems make it possible to measure the surface of a road when travelling it with a variable speed up to 90 km/h.

The various sub-systems are discussed in the next paragraphs.

![fig 11.1 The Automatic Road ANalyser.](image)

### 11.2.1.2 Longitudinal profile

The sub-system used to determine the longitudinal profile consists of two devices which measure vertical accelerations. The first one is situated on the centre of the rear axle, while the second one is placed on the chassis of the vehicle, above the one on the axle. The micro-computer processes the measured accelerations into profile heights. In this way it is possible to measure longitudinal unevenness within the wavelength area of 0.6 to 90 m ([46], [47]).

The ARAN, as used for this research, measured the vertical accelerations with such a frequency that each 200.55 mm a longitudinal profile height could be calculated by the micro-computer. These calculations are based upon a two mass spring simulation.

Since the device which measures the vertical accelerations of the rear axle is placed on the centre of this axle, the measured longitudinal profile has to be considered as a mean of the longitudinal profile in the right wheel track and the profile in the left wheel track. The average longitudinal profile might thus be smoother than the longitudinal profiles under each individual tire.
11.2.1.3 Transversal profiles

The system which is used to determine the transversal profile of a pavement consists of a cross bar with ultrasonic sensors. This bar is placed at the front of the vehicle, at a distance of about 4.25 m from the rear axle. It is situated about 0.4 m above the surface of the pavement and contains 19 sensors which are equally distributed over the length of the cross bar. The distance between two of these sensors is 0.1 m.

This 1.9 m long unremovable cross bar can be enlarged by connecting a 0.3 or 0.6 m long removable extra bar section at each side of it. In this way the maximal width over which measurements can be performed becomes 3.1 m.

By combining the measurements of the ultrasonic sensors with the measurements of a set of gyroscopes, see paragraph 11.2.1.4, a cross profile is obtained.

The rut depth of the left and right wheel track is determined by calculating the vertical distance between the lowest measured point in a wheel track and a line which connects the highest measured points at both sides of the wheel track, also see figure 11.2. The accuracy of this rut depth determined by the ARAN is about 1.5 mm (46, 48). Each 2 m these rut depths are determined.

The ARAN also measures complete transversal profiles each 4 m. This is done by combining the values measured by the various ultrasonic sensors over a 4 m distance. The combined ultrasonic sensor values are thus not all determined at the same transversal road profile, so that the measured transversal profiles do not really exist, they are formed by combination of measurements performed over a length of 4 m.

11.2.1.4 Gyroscopes

The ARAN is equipped with a set of three gyroscopes. Using these gyroscopes the longitudinal and cross slopes of a profile can be determined. The gyroscopes make it furthermore possible to determine the position of the cross bar at the front of the vehicle. Using this information the measurements of the ultrasonic sensors are adjusted.

11.2.1.5 Videologging

In order to be able to perform a visual condition survey, later on in the office, the ARAN is equipped with three video cameras. One of these cameras
delivers the view of the driver of the ARAN. The other two cameras, at the rear side of the vehicle, make close-up pictures of the pavement surface in the two wheel tracks.

The cameras used are fast enough (shutter time front camera 1/1,000 s, rear cameras 1/10,000 s) to give sharp pictures when driving with speeds up to 90 km/h.

The onboard computer of the ARAN adds important information to the pictures of the cameras. The most important information added are: the date, the time, the vehicle speed, the road chainage and codes referring to the measured road.

11.2.2 Rut depth

As mentioned earlier the ARAN measured the rut depth underneath a straight line, RDs, for the right and the left wheel track each 2 m, furthermore the ARAN determined a complete transversal profile each 4 m. From these transversal profiles of course the rut depth underneath a 1.2 m straight edge or relative rut depth, RDr, can be determined. In figure 11.2 the three types of rut depths RDs, RDl and RDt that are now distinguished in this dissertation are presented schematically.

Since the initial profile of the various pavements is not known RDs can not be retrieved from the ARAN data.

![Fig 11.2 The three different types of rut depths.](image)

In table 11.1 the RDl as determined by the ARAN for the seven test pavements are presented. In this table the average RDl as well as the observed standard deviation of RDl is given. A distinction is made between the left (inner) and the right (outer) wheel track.
<table>
<thead>
<tr>
<th></th>
<th>RD$_i$ in left wheel track</th>
<th></th>
<th>RD$_i$ in right wheel track</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>average [mm]</td>
<td>standard deviation [mm]</td>
<td>average [mm]</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>4.60</td>
<td>1.67</td>
<td>4.90</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>4.63</td>
<td>1.77</td>
<td>5.23</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>4.92</td>
<td>1.91</td>
<td>7.21</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>6.52</td>
<td>3.46</td>
<td>5.60</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>4.33</td>
<td>1.92</td>
<td>5.10</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>4.73</td>
<td>2.13</td>
<td>3.52</td>
</tr>
<tr>
<td>Weiver</td>
<td>26.69</td>
<td>9.54</td>
<td>17.69</td>
</tr>
</tbody>
</table>

*Table 11.1* RD$_i$ as measured by the ARAN.

Similar to table 11.1 the values of the average RD$_i$ and the standard deviation of RD$_i$ are presented in table 11.2.

<table>
<thead>
<tr>
<th></th>
<th>RD$_r$ in left wheel track</th>
<th></th>
<th>RD$_r$ in right wheel track</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>average [mm]</td>
<td>standard deviation [mm]</td>
<td>average [mm]</td>
</tr>
<tr>
<td>M. Havelaarweg</td>
<td>4.42</td>
<td>1.52</td>
<td>4.39</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>4.85</td>
<td>1.61</td>
<td>5.41</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>5.03</td>
<td>1.66</td>
<td>5.56</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>6.97</td>
<td>3.24</td>
<td>5.44</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>4.73</td>
<td>1.78</td>
<td>5.14</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>4.18</td>
<td>1.69</td>
<td>3.29</td>
</tr>
<tr>
<td>Weiver</td>
<td>26.10</td>
<td>8.75</td>
<td>17.48</td>
</tr>
</tbody>
</table>

*Table 11.2* RD$_r$ as determined on the basis of the transversal profiles measured by the ARAN.

As can be seen from figure 11.2 the value of RD$_r$ should be smaller than or equal to the value of RD$_i$. For the rut depths measured in the left wheel tracks of the Pascalweg and the Zaanweg this is not the case. For the Baarsweg and the C. Bruynweg RD$_r$ is larger than RD$_i$ for both the right and left wheel track.

The cause of these differences is not known. It is however stated that the differences remain limited (maximum 0.45 mm, left wheel track of the
Zaanweg) and far within the 1.5 mm accuracy of the ARAN rut depth measurements, so the differences might result from the limited measurement accuracy. Furthermore it is reminded that RD\text{r} is measured each 2 m while RD\text{r} is retrieved from transversal profiles measured over a 4 m pavement length, which explains that the two types of rut depths are based on somewhat different figures. This of course can also result in the small differences observed.

11.2.3 Longitudinal profiles

As mentioned the ARAN measured a longitudinal profile height each 200.55 mm. Of course plots can be made from these longitudinal profiles. An example of such a longitudinal profile plot is given in figure 11.3, which shows the profile that was measured on the Weiver in Zaanstad.

![Longitudinal profile as measured on the Weiver in Zaanstad.](image)

To get insight into the roughness of a pavement the measured longitudinal profile has to be analyzed. For this research hereto use is made of the Fourier analysis which is not discussed in this dissertation. To limit end-effects the measured longitudinal profiles were passed through a cosine taper data window before the actual Fourier analyses were performed (50).

As output the Fourier analysis gives a set of wavelengths and the corresponding amplitudes of unevenness present in the measured longitudinal profile. The further processing of this output is discussed hereafter.
The Fourier analyses performed on each of the measured longitudinal profiles consider 512 measured profile heights. Given the sampling distance of 200.55 mm this means that the length of the analyzed profile becomes 102.68 m. The wavelengths analyzed by the Fourier analyses now become

\[ \lambda_i = \frac{L}{i} = \frac{102.68}{i} \]  

(11.1)

where:
- \( \lambda_i \): wavelength of wave number "i" [m]
- L: length of analyzed signal [m]
- i: integer indicator [-]

The ARAN is able to measure unevenness with wavelengths between 0.6 m and 90 m. The largest i-value considered in the Fourier analyses thus becomes 171, so that the shortest wave considered equals 0.6005 m.

As mentioned the Fourier analyses gives the amplitudes of unevenness for the 171 considered wavelengths. To get a maximum insight into the roughness of a pavement it is necessary to translate the outcome of the Fourier analysis into a Power Density Spectrum, PDS (41, 40, 49). Such a spectrum gives the power density of the longitudinal profile as a function of the wavelength of unevenness. To explain what a PDS is, it is necessary to explain the term "power".

The energy present in a signal is determined by the integral of the squared profile height over the length of the signal, see equation 11.2. The energy present in a road section thus depends on both the roughness of the road section and its length, the longer the section the more energy it will contain. By dividing the energy in a road section by its length the power of that section is obtained, equation 11.3.

\[ E_t = \int_0^L y^2(x) \, dx \]  

(11.2)

\[ P_t = \frac{\int_0^L y^2(x) \, dx}{L} = \frac{E_t}{L} \]  

(11.3)

where:
- \( E_t \): energy in a road section [m³]
\( P_i: \) power in a road section \[ \text{[m}^2\text{]} \]

\( L: \) length of the road section \[ \text{[m]} \]

\( y(x): \) profile height as a function of the chainage "x" \[ \text{[m]} \]

\( x: \) distance from origin or chainage \[ \text{[m]} \]

The waves of unevenness as considered in the Fourier analyses are all sine shaped signals that exactly fit \( "i" \) times in the measured profile. Each of these sines of unevenness contribute to the total power of the road section.

\[
P_i = \frac{1}{2} A_{u,i}^2
\]

(11.4)

\[
P_t = \sum_{i=1}^{171} P_i
\]

(11.5)

where:

\( A_{u,i}: \) amplitude of longitudinal unevenness with wavelength \( \lambda_i \) \[ \text{[m]} \]

\( P_i: \) contribution of unevenness with a wavelength \( \lambda_i \) to the power of the road section under consideration \[ \text{[m}^2\text{]} \]

As indicated by equation 11.5 the total power of a road profile equals the sum of the power of unevenness with various wavelengths. The power density is the derivative of power to wavelength.

\[
A(\lambda) = \frac{dP_t}{d\lambda}
\]

(11.6)

On the basis of the wavelengths considered by the Fourier series equation 11.6 can be rewritten into equation 11.7.

\[
A(\lambda_i) = \frac{P_i}{\lambda_i - \lambda_{i+1}}
\]

(11.7)

Where:

\( A(\lambda_i): \) power density at a wavelength of \( \lambda_i \) (= 102.68/i) \[ \text{[m]} \]

On the basis of equation 11.7 the amplitudes determined by the Fourier analyses of the measured road profiles can easily be translated to power density. By plotting these power densities against the wavelength a PDS is obtained.

In analogy with other literature (41, 49) it was decided to plot the PDS
on a log-log scale and to use for this wavelengths which are equally spread over this scale. The amount of plotted points was for this research limited to 45. The shortest wavelength in the Fourier series equals 0.6005 m (log(\(\lambda\))=-0.221), while the longest wavelength equals 102.68 m (log(\(\lambda\))=2.011). Each of the 45 equally spread wavelengths thus considers a wavelength area of \((\frac{2.011 + 0.221}{45} =) 0.0496\) on the log(\(\lambda\)) scale.

The two largest wavelengths present in the Fourier series are 51.34 m and 102.68 m respectively. Within this wavelength area six plotted points can be found in the PDS. These six points are all based on the power density found at the mentioned two largest wavelengths. Hereto use is made of linear interpolation.

At shorter wavelengths the situation becomes the other way around. Now the points plotted in the PDS refer to more than one wavelengths in the Fourier series. The power density plotted at a wavelength of 4.97 m for instance considers the wavelength area of 4.695 m to 5.264 m. Within this wavelength area two wavelengths from the Fourier analysis are present, i.e. \(\lambda_{20}=5.13\) m and \(\lambda_{21}=4.89\) m. The power density plotted in these cases equals the average power density as found at the wavelengths analyzed by the Fourier series that are present within the wavelength area to which the plotted point refers.

In figure 11.4 the Power Density Spectrum determined for the longitudinal profile as measured on the Weiver in Zaanstad is presented. The straight line in figure 11.4 gives the results of a regression analysis of the plotted PDS. The equation of this regression line is presented in equation 11.8.

\[
A(\lambda) = a5m \cdot \left( \frac{\lambda}{\lambda_0} \right)^{ang}
\]  

(11.8)

where:

- a5m: regression parameter, power density at a wavelength of 5 m [m]
- ang: regression parameter, angle of the regression line [-]
- \(\lambda_0\): reference wavelength of 5 m [m]
- \(\lambda\): wavelength [m]

In analogy with other literature the power densities for wavelengths longer than 40 m are not used in the regression analysis of the PDS (41). It is clear that normal traffic is not so sensitive for such long wavelengths, so that the regression line is based upon the power density within the wavelength area for which normal road traffic is sensitive.
fig 11.4 Power Density Spectrum of longitudinal unevenness as measured for the Weiver in Zaanstad.

In appendix 11.1 the PDS of the seven concrete block pavements are presented. Equation 11.8 was fitted through the points plotted for wavelengths shorter than 40 m for all these spectra. In table 11.3 the parameters of the regression line through the PDS are presented for the seven pavements.

<table>
<thead>
<tr>
<th>Base</th>
<th>a5m [m]</th>
<th>ang [-]</th>
<th>r² [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>M. Havelaarweg</td>
<td>Yes 1245 . 10^9</td>
<td>1.10</td>
<td>0.74</td>
</tr>
<tr>
<td>Baarsweg</td>
<td>No 1992 . 10^9</td>
<td>0.79</td>
<td>0.58</td>
</tr>
<tr>
<td>Pascalweg</td>
<td>Yes 487 . 10^9</td>
<td>0.81</td>
<td>0.62</td>
</tr>
<tr>
<td>Zaanweg</td>
<td>No 1621 . 10^9</td>
<td>0.85</td>
<td>0.75</td>
</tr>
<tr>
<td>C. Bruynweg</td>
<td>Yes 847 . 10^9</td>
<td>0.78</td>
<td>0.53</td>
</tr>
<tr>
<td>Allanstraat</td>
<td>Yes 614 . 10^9</td>
<td>0.76</td>
<td>0.56</td>
</tr>
<tr>
<td>Weiver</td>
<td>No 1473 . 10^9</td>
<td>0.60</td>
<td>0.49</td>
</tr>
</tbody>
</table>

*table 11.3 Results of the regression analyses of the PDS.*
11.3 VERIFICATION AND VALIDATION

11.3.1 Introduction

In the previous section the results of road surface measurements were discussed. These results clearly show that the rut depth in a concrete block pavement is not a constant value, but that it shows a firm standard deviation, meaning that rut depth variations occur over the length of a pavement. Furthermore a concrete block pavement shows a significant longitudinal unevenness as shown by the PDS.

Both the rut depth variations and the effects hereof on longitudinal unevenness are explained by the theories discussed in the previous chapters, i.e. the model explaining the development of roughness due to dynamic axle loads. It is clear that in order to validate the theories that were developed in the previous chapters predictions made by these models should be verified on the basis of the profile measurements.

The absolute levels of rut depths can not be verified on the basis of the single profile measurements. The development of rutting, as explained by the rutting performance model, however is important input for the prediction of the development of longitudinal unevenness, and thus gives possibilities for verification and validation.

The verification approach that was used was therefore as follows. A prediction is made with the essential models of the rut depth development in time and its variation. As the mean rut depth increases also the variation will increase. A way to verify the models is to compare the relation, rut depth with a 30% probability of exceeding versus standard deviation of the rut depth, as derived from the field data with a similar relation developed from the theoretical analysis. Furthermore the roughness development as observed in the field can be compared by the one predicted by means of the models.

If the predictions fit well with the observations then it can be concluded that the models can predict in-situ conditions well.

It is remarked here that the behaviour of a concrete block pavement strongly depends on the properties of traffic (i.e. lateral wander, contact pressures, axle loads, dynamical properties and speeds). Since most of these properties are not known exactly, the verification remains a relative one since only trends can be compared. A pure one-to-one (absolute) verification is not possible.
11.3.2 Calculations

In order to verify the developed models, two calculations are made by means of the rutting performance model and the model explaining the development of longitudinal unevenness due to rut depth variations. In both calculations the traffic contains 96% "passenger car 1", see chapter 10. Half of these cars travel at 45 km/h and the remaining cars travel at 55 km/h.

The remaining 4% of traffic consists of "truck 1, 100 kN", see chapter 10. Half of these trucks have a speed of 30 km/h and the other trucks travel at 45 km/h.

In both calculations traffic has a standard deviation of lateral wander of 200 mm, all loads are applied to the pavement with a 700 kPa contact pressure.

The first calculation refers to a pavement with only a 600 mm thick Zaanweg sand sub-base over the 60 MPa subgrade. For this pavement the rutting performance model gives the following parameters: $a_p=5.97$ [mm], $b_p=0.33$ [-], $c_p=0$ [mm], $d_p=0$ [-], $A_{sl}=98.78$ [kN], $m_l=6.29$ [-], $\sigma_{c,sl}=700$ [kPa], $m_c=2.33$ [-] and $R_P=1.39$ [-]. If the allowable relative rut depth, $RD_r$, is taken as 15 mm, this pavement has a design life of 15,853 standard axle load repetitions (331,653 vehicle repetitions) when loaded by the static axle loads in traffic.

The second calculation refers to a pavement with a 50 mm bedding layer over a 250 mm Max Havelaarweg base and a 600 mm Zaanweg sand sub-base. Again the pavement is placed over a 60 MPa subgrade. The rutting performance model for this pavement gives the following parameter values: $a_p=2.16$ [mm], $b_p=0.033$ [-], $c_p=0.045$ [mm], $d_p=0.0090$ [-], $A_{sl}=99.50$ [kN], $m_l=11.79$ [-], $\sigma_{c,sl}=700$ [kPa], $m_c=4.74$ [-] and $R_P=1.25$ [-]. Assuming again $RD_r=15$ mm, the design life of this pavement is 626,437 standard axle load repetitions (14,568,302 vehicle repetitions) when the pavement is loaded by the static axle loads of traffic.

In figure 11.5 the calculated development of the relative rut depth, $RD_r$, for the two concrete block pavements is presented. In order to show the results of both calculations in one figure the design life of the pavements is made relative by assuming a 100% design life when the $RD_r$ with a 30% probability of exceeding equals 15 mm.

As can be concluded from figure 11.5, over the first 80% of its design life (which in absolute terms is much greater for the pavement with a base layer when compared to the pavement with a sand sub-base only) the pavement with a base layer is much less sensitive for developing damage as the pavement with a sand sub-base only. During this first 80% the pavement with a base layer hardly develops rut depth, whereas the pavement with a sand sub-base shows a steady accumulation of rut depth over its entire design life.
During the last 20% of its design life the pavement with a base shows a very strong accumulation of rut depth. In this 20% of the design life of the pavement with a base the standard deviation of the rut depth shows a strong increase, implying that the pavement is now rapidly accumulating roughness. This is clearly shown by figure 11.6 which gives the development of the maximum value of the $Au_n[N]/Au_n[0]$-ratio for the two pavements.

**Fig 11.5** Development of the rut depth in the two analyzed pavements.

**Fig 11.6** Development of the maximum value of the $Au_n[N]/Au_n[0]$-ratio for the two analyzed pavements.
**Fig 11.7** Development of the relative amplitude spectrum for the pavement with a sub-base only.

**Fig 11.8** Development of the relative amplitude spectrum for the pavement with a base layer.

The figures 11.7 and 11.8 are presented to give a better insight into the wavelengths at which traffic amplifies and smoothes longitudinal unevenness. In these figures the development of the relative amplitude spectra for the two pavements under the mentioned traffic are presented. In the figures the number of vehicle passages for which the relative amplitude spectra are determined are indicated.

Again these figures show that the pavement with a base is hardly
accumulating damage during the first major part of its design life, at long term however the pavement with a base layer accumulates damage at a rapid rate.

The pavement with a sand sub-base only does not show this rapid accumulation of damage during the last part of its design life. This pavement shows a more steady accumulation of damage over its complete design life.

11.3.3 Comparison of the calculations and the measurements

In the previous section a very brief discussion of the calculation results was given. The results of these two calculations can at some points be compared with the results of the profile measurements.

11.3.3.1 Rut depth

The first comparison that can be made is the comparison of the measured and calculated relations between RD, with a 30% probability of exceeding and the standard deviation of RD,. In figure 11.9 the results of this comparison are presented.

![Figure 11.9](image)

*Fig 11.9 Calculated and measured relation between standard deviation of RD, and RD, with a 30% probability of exceeding.*

The following comments are made with respect to the differences in the observed and calculated relation between RD, with a 30% probability of
exceeding and the standard deviation of $RD_i$.

First of all it should be stated that the calculations only consider rut depth variations due to the variations of the axle loads that are applied to a pavement. In reality rut depth variations can develop due to numerous causes, axle load variations as a result of the dynamic response of vehicles to roughness are only one of them.

Rut depth variations can for instance also develop when the pavement rutting behaviour varies over the length of the pavement. This can be a result of varying layer thicknesses or varying properties of the materials in the substructure either due to variations in compaction level, variations in moisture content or variations in grading, grain shape or grain material.

Since there are numerous causes for rut depth variations, it is expected that the variations, i.e. the standard deviation of the rut depth, that are explained by the model (variations due to dynamic vehicle responses) remain smaller than the variations that are measured in reality.

As is shown by figure 11.9 the calculation for the pavement with a sand sub-base only complies with the expectation. This calculation shows that the calculated standard deviation of $RD_i$ equals about 0.25 times the measured standard deviation.

The results that are obtained for the pavement with a 250 mm base do not completely comply with the expectations. At lower levels of accumulated rut depth, values smaller than about 4.5 mm, the results of the computation show that the computed standard deviation of rut depth is smaller than the measured standard deviation, as expected. With a further increase of the level of accumulated rut depth the calculated standard deviation of $RD_i$ however becomes larger than the measured standard deviation, which does not comply with the expectations.

It should be noted however that according to figure 11.5 a 4.5 mm rut depth with a 30% probability of exceeding in this pavement only occurs when the pavement has reached about 80% of its entire life. So one can conclude that over that period the models for pavements with a base layer seem to work correctly.

It is believed that the deviations that occur in the last 20% of the pavement life are a result of the simplified concrete block pavement behaviour on which the roughness model is based. The roughness model is based upon the linearized behaviour of the pavement under the static wheel loads in traffic. Of course the actual pavement behaviour is not linear. Due to axle load variations the average rut depth that develops is larger than the rut depth that develops under the average or static axle loads. This phenomenon is not accounted for in the roughness model. As a result hereof the development of the average rut depth as determined by the model is less than determined on the basis of the non-linearized pavement behaviour.

The effects of using the linearized pavement behaviour as the basis of
the model become especially strong when the rutting behaviour of the pavement becomes extra sensitive for the loads applied to the pavement. This is the case for pavements with a base layer that show a large value of $I_m$ (load equivalency coefficient). For these pavements the development of the average rut depth as explained by the linearized pavement behaviour is less than based on the actual pavement behaviour. As a result the relation between the standard deviation of rut depth and the rut depth with a 30% probability of exceeding will for these pavements be exaggerated.

The effects of the linearized rutting behaviour become especially strong during the last 20% of the design life of the pavement with a base. In this part of the pavements life the accumulated rut depth becomes very dependable on magnitude of the axle loads applied to the pavement. During these last 20% of the design life of a pavement with a base layer, the average rut depth explained by the roughness model will as a result be far less than the average rut depth based on the actual, non linearized, pavement behaviour under the dynamic axle loads.

11.3.3.2 Roughness

Apart from the standard deviation of the rut depth also the development of roughness can be compared with the measured roughness. This comparison requires somewhat more explanation as is discussed hereafter.

As stated more often in this dissertation, it is assumed that the average PDS of a natural, not trafficked, longitudinal road profile equals a straight line on a log-log scale. If traffic effects the roughness in a road profile as explained by the models discussed in this dissertation, the PDS of a road profile will change during trafficking, implying that the PDS will show more and more differences with the straight line on a log-log scale.

To get insight into the effects of real traffic on roughness, the initial longitudinal profiles are needed. For this research such initial profiles are not available. It is however expected that on average an initial longitudinal profile will result in a straight line PDS on a log-log scale. Since traffic, according to the models, amplifies uneveness at some wavelengths while it smoothens unevenness at other wavelengths, it can be expected that traffic induced longitudinal unevenness will not so much effect the regression line through the PDS during trafficking. Due to traffic induced longitudinal unevenness, mainly the correlation of the fit between the line and the PDS will be effected (show a decrease in time).

This hypothesis can be proven by comparing the PDS regression lines
that have been developed for the various pavements, see appendix 11.1. Figure 11.10 shows these lines for the seven measured pavements. It can be observed that the lines found for pavements with a sand sub-base only are very close together. For pavements with a base layer it is shown that again 3 of the 4 regression lines are very close together. Only the Max. Havelaarweg is clearly deviating from the others.

Fig 11.10 PDS regression lines for the seven measured pavements.

Figure 11.10 learns that in general the regression lines for pavements with a sand sub-base only indicate a higher initial roughness level on these pavements than on pavements with a base layer. This is expected since a base layer is constructed mechanically, compacted and accurately levelled out. This implies that the pavement has a constant reference level about 50 mm (thickness of the bedding layer) underneath the sand he is working in. As a result the pavement will be able to construct a smoother block layer.

After construction of the block layer the jointing sand and bedding layer is compacted using a vibrating plate. In the case of a base layer this compaction only affects a 50 mm thick (bedding) layer, so that unevenness due to a varying density level of the compacted bedding sand results in only limited roughness after compaction.

In the case of a pavement without a base layer there is no layer that can act as a reference level to the pavement, furthermore compaction by means of a vibrating plate of the block layer now affects more than the upper 50 mm of the sand substructure, so that a larger roughness after compaction might be the result.
Based on the previous discussion it is thus expected that the average initial PDS equals a straight line on a log-log scale. This line is determined by the regression line through the average PDS of the measured test pavements. Of course the straight line through the average PDS does not completely comply with the actual average PDS. The differences between the straight line and the actual PDS are assumed to be a result of traffic effects on pavement roughness.

The differences between the average PDS and the straight line through it can be translated to a relative amplitude spectrum, which is given in figure 11.11.

\[ \frac{A_{u_1}}{A_{u_0}}[\text{]} \]

\text{pavement 1: sand sub-base only}
\text{pavement 2: base and sand sub-base}
\text{N = number of vehicle repetitions}

\text{measured (average)}
\text{calculated for pavement / N:}
\text{2 / 7,500,000}
\text{1 / 300,000}
\text{2 / 8,250,000}
\text{1 / 875,000}

\text{fig 11.11} \hspace{0.5cm} \text{Relative amplitude spectra as determined from the measurements and as calculated.}

In figure 11.11 in total 5 relative amplitude spectra are presented. The four smooth spectra are determined on the basis of model calculations while the irregular spectrum is determined on the basis of the measurements, as described.

The model calculations give the development of the relative amplitude spectrum as shown by the figures 11.7 and 11.8. In figure 11.11 only two relative amplitude spectra for each calculation are given. For the calculation that considers the pavement with a sand sub-base only (pavement 1) the relative amplitude spectra after 300,000 and 875,000 vehicle repetitions are given. These spectra are chosen such that the maximum amplification at a wavelength of 10 m is in one case somewhat smaller than measured and in the
other case somewhat larger than measured.

Similar to this two calculated spectra for the pavement with a base layer (pavement 2) are plotted. Again these spectra are chosen such that one of them shows a slightly larger amplification than measured at a wavelength of 10 m, while the second spectrum shows a somewhat smaller peak value at a wavelength of 10 m.

By considering figure 11.11 it becomes very clear that the effects of traffic on roughness as modelled closely equal the effects of traffic on roughness as determined on the basis of the measurements. Within two wavelength areas however substantial differences are observed.

First of all the relative amplitude spectra as determined by the models do not equal the spectrum as determined on the basis of the measurements for wavelength longer than about 40 m. This is a result of the fact that axle load variations at such wavelengths remain only limited. As a result traffic (travelling at normal speeds) hardly effects roughness in that particular wavelength area, as indicated by the calculated spectra. The unevenness of these wavelengths is usually caused by subgrade settlements which are not considered in this research.

The second wavelength area in which significant differences are found is the wavelength area of wavelengths shorter than about 1.4 m. Within this wavelength area the model shows a stronger smoothing effect of traffic than indicated by the spectrum based on the measurements.

It is expected that this is a result of the fact that the model gives insight into the development of rut depth variations and thus longitudinal unevenness due to dynamic axle loadings only. As discussed there are however other factors that will also result in rut depth variations and thus longitudinal unevenness such as variations in layer thickness, compaction level and material quality. These factors all result in a pavement rutting behaviour varying over the pavement length. Since the rutting behaviour of a concrete block pavement is largely determined in the upper 1 m of the substructure it is expected that these factors have a strong effect on rut depth variations and longitudinal unevenness with wavelengths up to about 1 m.

The effects of the variation of the rutting behaviour over the length of a pavement on the development of roughness are however not taken into account in this research. As a result the smoothing effects of traffic are not compensated by the roughness introducing effects of a varying rutting behaviour for wavelengths shorter than about 1.4 m, which explains the differences between the calculations and the measurements in this wavelength area.
Apart from the differences found for wavelengths shorter than about 1.4 m, figure 11.11 clearly shows that the model is very well capable in explaining the effects of traffic on the development of roughness. The shapes of the plotted relative amplitude spectra closely resemble each other and the trends found by the model comply with the trends found on the basis of the measurements.

11.3.4 Conclusions

Based on the previous sections it is concluded that traffic induced rut depth variations and their effect on the longitudinal profile are properly explained by the roughness model.

In reality, however, rut depth variations are also caused by variations in layer thicknesses, compaction levels, material quality etc. The effects of these variations are not considered in the presented model. This explains the differences between predicted and observed roughness at wavelengths shorter than about 1.4 m.

The linearized rutting behaviour, on which the model explaining the development of longitudinal unevenness is based, results in an average rut depth development that is somewhat smaller than the development of the average rut depth as explained by the non linearized behaviour. The effects of this simplification become especially strong during the last 20% of the design life of pavements with a base layer. In this part of the design life the rut depth under static loadings (average rut depth in roughness model) is smaller than the average rut depth based on the dynamic loadings and the actual pavement's rutting behaviour.
Appendix 11.1

Power Density Spectra of longitudinal profiles

In this appendix the Power Density Spectra of the seven in-service concrete block pavements, on which profile measurements are performed, are presented. Each figure shows the actual Power Density Spectrum combined with a regression line through the points plotted for wavelengths shorter than 40 m. The equation for this line is given in equation 11.8, while table 11.3 gives the results of the regression analyses.

\[ \text{fig a11.1.1 PDS as determined for the longitudinal profile of the Zaanweg.} \]
Fig a11.1.2 PDS as determined for the longitudinal profile of the Weiver.

Fig a11.1.3 PDS as determined for the longitudinal profile of the Allanstraat.
**fig a11.1.4** PDS as determined for the longitudinal profile of the C. Bruynweg.

**fig a11.1.5** PDS as determined for the longitudinal profile of the Baarsweg.
\textbf{fig a11.1.6} PDS as determined for the longitudinal profile of the M. Havelaarweg.

\textbf{fig a11.1.7} PDS as determined for the longitudinal profile of the Pascalweg.
Conclusions and recommendations

12

12.1 INTRODUCTION

In this final chapter the main conclusions of this thesis work are summarized and some recommendations are given. The conclusions and recommendations are given in three parts. First of all those related to the modelling work will be given. They are of special interest to the researcher. Following this, conclusions and recommendations related to the structural design of concrete block pavements are given. These are of special interest to the consultant responsible for designing concrete block pavements. Finally the conclusions and recommendations related to the construction of concrete block pavements are summed-up. These should be of interest for road authorities and contractors.

12.2 THEORETICAL CONCLUSIONS AND RECOMMENDATIONS

Numerous conclusions and especially recommendations about the developed models and theories can be drawn. In this paragraph only the most important ones are given:

* It is concluded that the structural finite element model for the resilient analysis functions well. The model respects the stress-dependent behaviour of the applied materials and takes into account the properties of the concrete block layer. As a result the model computes realistic sub-structure stresses due to a wheel loading and thus enables the determination of the development of permanent sub-structure strain and surface deformation that follows from it.

* The finite element model used in this research is an axial symmetric model. Although special attention was paid to the axial symmetric representation of the block layer, a 3D model will remain the ultimate geometrical model for the 3D concrete block layer.

* The straightforward repeated vehicle simulation results in a very good description of the effects of dynamical loadings on the development of permanent surface (transversal and longitudinal) deformation in concrete
block pavements. This approach is however very time consuming.

* The simplified roughness model based on the linearized rutting behaviour of a concrete block pavement is much more practical since it does not require repeated vehicle simulation and gives results which in general are in good agreement with the repeated vehicle simulation.

* This simplified roughness model however does not take into account the effects of dynamical loadings on the development of the average rut depth. Especially in the case that the rut development in a pavement shows a very strong dependency on the axle load this shortcoming of the model results in less accurate results. It is therefore recommended for future research to expand the model in such a way that the secondary phenomenon of an increasing average rut depth as a result of dynamical loadings is taken into account.

* Within this research no attention was given to the effects of a varying rutting behaviour over the length. This varying rutting behaviour can be a result of numerous causes such as; varying layer thicknesses, varying compaction levels, varying grain shapes, varying grading, varying grain material, etc. In this research it is shown that a varying rutting behaviour results in the introduction of rut depth variations and related longitudinal unevenness with wavelengths shorter than about 1.4 m. In order to come to a more complete description of the behaviour of concrete block pavement under traffic it is thus recommended to take also into account the effects of a varying rutting behaviour in future research.

12.3 CONCLUSIONS AND RECOMMENDATIONS FOR DESIGN

The most important design conclusions and recommendations that can be drawn on the basis of the work presented are as follows:

* Concrete blocks rotate and translate when a concrete block pavement is loaded by a wheel load. As a result concrete block pavements can only be analyzed by means of models that are able to simulate these phenomena and consider the discontinuities in the toplayer.

* The effects of implementing the stress-dependent behaviour of granular materials on the calculated deflection bowls is only minimal, on the calculated stresses these effects are however very significant. Designing a concrete block pavement on the basis of substructure stresses is thus only possible if the stress-dependent behaviour of granular materials is taken into account.

* All this means that the use of linear-elastic multi-layer models should be discouraged.
* The name assigned to a material, such as "sand" or "crushed concrete/masonry mix", does not describe the material behaviour. It is thus not possible to design a concrete block pavement on the basis of only such material descriptions. The use of descriptions which represent the most important mechanical material characteristics is therefore strongly recommended.

* The behaviour of a concrete block pavement is not only determined by the substructure design and the used materials. The weight and the dynamic effects of traffic also strongly affect the behaviour of a concrete block pavement. Designing a concrete block pavement without taking into account these effects might lead to major design errors and should thus not be practised.

* Vehicle dynamics cause smoothing and amplification of unevenness combined in one process. The wavelengths at which both types of vehicle effects are observed however differ with the traffic properties. The peak values of the smoothing effects and amplifying effects differ with both traffic properties and pavement behaviour. This means that in the design the traffic spectrum should be taken into account.

* It has no use to design thin base course layers in concrete block pavement structures. The minimum base layer thickness required in order to get a strong improvement of the pavement's behaviour varies somewhat with traffic and the type of base material. In general however a base layer should not have a thickness less than 250 mm.

12.4 CONCLUSIONS AND RECOMMENDATIONS FOR CONSTRUCTION

* The concrete block layer will only obtain a certain load spreading capability in those cases that joint forces can develop. To obtain a block layer with stiff joints (i.e. a block layer with an optimal load spreading capability) it is recommended to construct a block layer with narrow (2 - 3 mm) joints. It is important that these narrow joints will be properly sand filled. Here to a vibrating compaction plate can be used to compact the concrete block layer including jointing sand.

* The edge restraint is a very important part of a concrete block pavement. Only in the case of edge restraints that are rigid enough, joint normal forces can develop. If such an edge restraint is not applied, block rotation will result in block layer expansion, eventually leading to a complete loss of block pattern and widening (weakening) of the joints.

* The development of rut depth variations over the length of the pavement is strongly related to longitudinal unevenness. As a result the amount of roughness in the initial longitudinal profile is of influence on the

Conclusions and recommendations 405
pavement design life. It is thus recommended to construct a concrete block pavement as smooth as possible, especially in those wavelength area in which traffic amplifies roughness (about 1.5 m to 4 m and about 10 m to 40 m for normal traffic).

* Longitudinal unevenness is not only introduced by the effects of dynamic axle loadings, but also as a result of varying rutting characteristics over the length. To limit the roughness development it is recommended that the pavement should be constructed with the utmost care for quality;
  - Layer thicknesses must be constant over the length of the pavement.
  - The compaction level of the various layers must be constant over the length of the pavement. This implies that it is not allowed to use a vibration plate to diminish surface roughness due to a poor construction.
  - The quality of the materials (grading, grain shape, grain material, etc.) should be as constant as possible.
  - Given the strong effect of a base layer on the pavement rutting behaviour the previous points are especially important for the construction of the base layer, if any.

* The development of rutting and longitudinal unevenness due to dynamic axle loadings heavily depends on the behaviour of the applied materials. The behaviour of granular materials heavily depends on the compaction level. To obtain a high quality concrete block pavement it is thus recommended to pay a lot of attention to material compaction. Since rutting is mainly a result of permanent strains in the upper 0.5 m of the substructure it is of special importance to heavily compact this upper part of the substructure.

* The development of longitudinal unevenness depends on the properties of both the pavement and traffic. If traffic is very homogeneous (for instance public transport busses), danger of traffic induced roughness is very large. In such cases the previously mentioned recommendations referring to the development of roughness become especially valid.
List of symbols

a: parameter which relates $U_p$ to $U_r$ or $\epsilon_p$ to $\epsilon_r$ [-]

a: acceleration [m/s$^2$]

a: CBR actuator force at 2.54 mm penetration [N]

a$_{cw}$: vertical acceleration of centre of gravity of carriage work [m/s$^2$]

a$_{sv}$: regression parameter found by Volders and Verhoeven [s/g]

a$_{sp}$: regression parameter found by Volders and Verhoeven [s/g]

ang: angle of the power density spectrum on log-log scale [-]

a$_p$: parameter describing the rut development in a pavement [mm]

arm: length of the arm over which a force acts [m]

a$_{v1}$: vertical acceleration of the front axle including tires [m/s$^2$]

a$_{v2}$: vertical acceleration of the rear axle including tires [m/s$^2$]

a1: parameter that relates A to stresses [-]

a2: parameter that relates A to stresses [-]

a$_{5m}$: power density at a wavelength of 5 m [m]

A: parameter relating permanent strain to number of load repetitions [-]

A$_{ff_{\lambda,i}}$: amplitude of dynamic component of front axle load of vehicle "$i" with wavelength "$\lambda" [kN]

A$_{fr_{\lambda,i}}$: amplitude of dynamic component of rear axle load of vehicle "$i" with wavelength "$\lambda" [kN]

A$_i$: amplitude of sine number "$i" [unit depends on what sine represents]

AL$_{st}$: standard axle load (= 2 x L$_{st}$) [kN]

A$_{u\lambda}$: amplitude of longitudinal unevenness with wavelength $\lambda$ [mm]

Arda$_{\lambda}$: amplitude of the sine representing the absolute rut depth variations with a wavelength "$\lambda" [mm]

b: parameter which relates $U_p$ to $U_r$ [-]

b: CBR actuator force at 5.04 mm penetration [N]

b$_{sv}$: regression parameter found by Volders and Verhoeven [s]

b$_{sp}$: regression parameter found by Volders and Verhoeven [s]

b$_p$: parameter describing the rut development in a pavement [-]

b1: parameter that relates B to stresses [-]

b2: parameter that relates B to stresses [-]

B: parameter relating permanent strain to number of load repetitions [-]

c: cohesion [kPa]

c$_{cf}$: combined damper constant of the front shock absorbers [Ns/m]

c$_{sr}$: combined damper constant of the rear shock absorbers [Ns/m]
$c_{p_i}$: parameter describing the rut development in a pavement [mm]
$c_{pa_i}$: parameter describing the effect of $a_p$ on $c_p$ [-]
$c_{pa_i}$: parameter describing the effect of $a_p$ on $c_p$ [-]
$c_{pb_i}$: parameter describing the effect of $a_p$ on $b_p$ [-]
$c_{pb_i}$: parameter describing the effect of $a_p$ on $b_p$ [-]
$c_{cp_i}$: parameter describing the effect of $c_p$ on $c_p$ [-]
$c_l$: parameter that relates C to stresses [-]
$c_{2}$: parameter that relates C to stresses [-]
$C$: parameter relating permanent strain to number of load repetitions [-]
$CBR_i$: Californian bearing ratio [-]
$CBR_{1i}$: CBR at 2.54 mm penetration [-]
$CBR_{2i}$: CBR at 5.08 mm penetration [-]
$Cf_{R,i}$: combined cosine term in roughness model taking into account $Cf_{R,i}$ and $Cr_{R,i}$ [-]
$Cf_{R,i}$: cosine term in roughness model for front axle load variations with a wavelength "$\lambda$" induced by vehicle "i" [-]
$Cr_{R,i}$: cosine term in roughness model for rear axle load variations with a wavelength "$\lambda$" induced by vehicle "i" [-]
$d_{e,wa}$: distance between the centre of gravity of the unloaded carriage work and the centre of gravity of the carriage work including payload [m]
$d_{i}$: distance between the centre of gravity of the payload and the centre of gravity of the carriage work including payload [m]
$d_{p}$: parameter describing the rut development in a pavement [-]
$d_{0}$: 50 kN standard load deflection in load centre [$\mu$m]
$d_{0,3}$: 50 kN standard load deflection 0.3 m from load centre [$\mu$m]
$d_{1}$: parameter that relates D to stresses [-]
$d_{2}$: parameter that relates D to stresses [-]
$d_{\max}$: initial resilient vertical deformation of a layer under a 80 kN standard axle load [mm]
$D$: parameter relating permanent strain to number of load repetitions [-]
$D_{i}$: diameter of spheres [mm]
$D_{l}$: design life [-]
$D_{l,i}$: design life of a pavement under the wheel load spectrum in total [-]
$D_{l,i}$: design life of a pavement under wheel load "i" in the wheel load spectrum [-]
$D_{\max}$: maximum diameter of the spheres [mm]
$e_{a_{R,i}}$: parameter describing $a_p$ [-]
$e_{c_{R,i}}$: parameter describing $c_p$ [-/MPa]
$e_{c_{R,i}}$: parameter describing $c_p$ [-]
$E$: Young’s modulus [MPa]
$E_{o}$: stiffness of the subgrade [MPa]
$E_{0}$: reference subgrade modulus of 100 MPa [MPa]

List of symbols
ff, A/Au - ratio \[\text{[kN/mm]}\]
fr, A/Au - ratio \[\text{[kN/mm]}\]
F: \[\text{[N]}\]
F_{eff}: dynamical vertical force in front shock absorbers \[\text{[N]}\]
F_{ext}: dynamical vertical force in rear shock absorbers \[\text{[N]}\]
F_{kfr}: dynamical vertical force in front suspension springs \[\text{[N]}\]
F_{krs}: dynamical vertical force in rear suspension springs \[\text{[N]}\]
F_{kfr}: dynamical vertical force in front tires \[\text{[N]}\]
F_{ktr}: dynamical vertical force in rear tires \[\text{[N]}\]
F_{str}: static vertical force in front suspension system \[\text{[N]}\]
F_{stt}: static vertical force in rear suspension system \[\text{[N]}\]
F_{str}: dynamical vertical force in front suspension system \[\text{[N]}\]
F_{str}: dynamical vertical force in rear suspension system \[\text{[N]}\]
F_{stt}: static vertical force in front tires \[\text{[N]}\]
F_{stt}: static vertical force in rear tires \[\text{[N]}\]
g: gravitational acceleration \[\text{[m/s}^2\]\]
i: integer indicator \[\text{[-]}\]
i_{\text{max}}: maximum value of i \[\text{[-]}\]
D_{h, j}: surface curvature index \([d_0 - d_{0.3}]\) \[\text{[\mu m]}\]
IRI: international roughness index \[\text{[mm/m]}\]
J: moment of inertia \[\text{[kg m}^2\]\]
J_{cw}: moment of inertia of the carriage work including payload, if any \[\text{[kg m}^2\]\]
J_{cw}: moment of inertia of the unloaded carriage work \[\text{[kg m}^2\]\]
J_{i}: moment of inertia of the payload \[\text{[kg m}^2\]\]
k_{fr}: combined stiffness of the front suspension springs \[\text{[N/mm]}\]
k_{sr}: combined stiffness of the rear suspension springs \[\text{[N/mm]}\]
k_{tr}: total stiffness of the front tires \[\text{[N/mm]}\]
k_{tr}: total stiffness of the rear tires \[\text{[N/mm]}\]
k_1: parameter relating stiffness to stresses \[\text{[MPa]}\]
k_2: parameter relating stiffness to stresses \[\text{[MPa]}\]
k_5: parameter relating stiffness to stresses \[\text{[-]}\]
k_6: parameter relating stiffness to stresses \[\text{[-]}\]
k_7: parameter relating stiffness to stresses \[\text{[-]}\]
k_8: parameter relating stiffness to stresses \[\text{[-]}\]
l_{cw}: horizontal distance between the front axle and the centre of gravity of the carriage work including payload, if any \[\text{[m]}\]
l_{cw}: horizontal distance between the front axle and the centre of gravity of the unloaded carriage work \[\text{[m]}\]
l_{c}: horizontal distance between the front axle and the centre of gravity of the payload \[\text{[m]}\]
l_{c}: horizontal distance between the rear axle and the centre of gravity of the carriage work including payload, if any \[\text{[m]}\]
lwa₁: parameter describing the effect of $\sigma_{tw}$ on $a_p$ [-]
lwa₂: parameter describing the effect of $\sigma_{tw}$ on $a_p$ [-]
lwb₁: parameter describing the effect of $\sigma_{tw}$ on $b_p$ [-]
lwb₂: parameter describing the effect of $\sigma_{tw}$ on $b_p$ [-]
lwc₁: parameter describing the effect of $\sigma_{tw}$ on $c_p$ [-]
L: wheel load [kN]
La₁: parameter describing the effect of L on $a_p$ [-]
La₂: parameter describing the effect of L on $a_p$ [-]
Lb₁: parameter describing the effect of L on $b_p$ [-]
Lb₂: parameter describing the effect of L on $b_p$ [-]
Lc₁: parameter describing the effect of L on $c_p$ [-]
Lf: static front axle load of vehicle "i" in traffic [kN]
Lq: magnitude of wheel load "i" [kN]
LL₁: grain size at lower limit of fraction [mm]
LL₂: grain size at upper limit of fraction [mm]
Lr₁: static rear axle load of vehicle "i" in traffic [kN]
Lr: reference 50 kN wheel load [kN]
Lₘ: standardized wheel load [-]
Lₘₛ: standard wheel load [kN]
Lₘₛ₁: static front axle load [kN]
Lₘₛ₂: static rear axle load [kN]
mc: contact pressure equivalency coefficient [-]
mₗ: load equivalency coefficient [-]
M: mass [kg]
Mₑₑ: mass of carriage work including payload, if any [kg]
Mₑₑₚ: mass of carriage work without payload [kg]
Mₑ: mass of the payload [kg]
Mₑₑ₁: mass of the front axle including tires [kg]
Mₑₑ₂: mass of the rear axle including tires [kg]
Mr: resilient modulus [MPa]
Mrₑₑ₁: resilient modulus in element "e" for following iteration [MPa]
Mrₑₑ₂: resilient modulus in element "e" on the basis of the calculated stresses [MPa]
N: number of load repetitions [-]
Nₑₑ₁: number of axle load repetitions [-]
Nₑₑₚ: number of equivalent standard axle load repetitions [-]
Nₑₑ₂: $N_{ae}$ at which progressive stiffening starts [-]
Nₑₑ₃: standard wheel load repetitions caused by a passage of a fictitious wheel load in which all wheel loads in the spectrum are considered [-]
Nₑₑ₄: number of load repetitions with specified properties [-]
Nₑₑ₅: equivalent number of standard axle or wheel load repetitions [-]
Nₑₑ₆: lower limit of the window in standard axle load repetitions for
which the development of longitudinal unevenness is determined
upper limit of the window in standard axle load repetitions for
which the development of longitudinal unevenness is determined
number of standard wheel load repetitions divided by 1,000 in
order to come to RD₁
number of standard wheel load repetitions divided by 1,000 in
order to come to RD₂
number of standard wheel load repetitions divided by 1,000 in
order to come to RD₃
number of standard wheel load repetitions divided by 1,000 in
order to come to RD₄
fictitious outflow time of WF [g] of glass pearl shaped grains
average outflow time of weight quantity under consideration
constant depending on layer thickness and layer material
mass percentage of excluded fraction for dry material
corrected Proctor density
percentage of a fraction
mass percentage of excluded fraction for wet material
constant depending on layer thickness and layer material
radius of the wheel load area
parameter controlling iteration process
parameter controlling iteration process
rut depth
absolute rut depth
relative rut depth (rut depth underneath a 1.2 m straight edge)
RDₐ at failure or relative rut depth design criterion
relative rut depth level of 0.25 x RDₐₜ
relative rut depth level of 0.5 x RDₐₜ
relative rut depth level of 1 x RDₐₜ
relative rut depth level of 1.5 x RDₐₜ
RDₐ/RDₐₜ-ratio
standard deviation
combined sine term in roughness model taking into account Sfₐ,i
and Srₐ,i
sine term in roughness model for front axle load variations with a
wavelength "λ" induced by vehicle "i"
sine term in roughness model for rear axle load variations with a
wavelength "λ" induced by vehicle "i"
average specific weight of sand grains ( =2.65)
specific weight of the grains in a granular material
time
parameter describing aₚ
parameter describing aₚ

List of symbols
\( t_{a_1} \):  parameter describing \( a_p \) 
\( t_{a_2} \):  parameter describing \( a_p \) 
\( t_{a_3} \):  parameter describing \( a_p \) [mm] 
\( t_{a_4} \):  parameter describing \( a_p \) [mm] 
\( t_{a_5} \):  parameter describing \( a_p \) [mm] 
\( t_{a_6} \):  parameter describing \( a_p \) [mm] 
\( t_{a_7} \):  parameter describing \( a_p \) [-] 
\( t_b \):  thickness of the base layer [mm] 
\( t_b^\prime \):  parameter describing \( b_p \) [-] 
\( t_{b_1} \):  parameter describing \( b_p \) [-] 
\( t_{b_2} \):  parameter describing \( b_p \) [-] 
\( t_{b_3} \):  parameter describing \( b_p \) [-] 
\( t_{b_4} \):  parameter describing \( b_p \) [-] 
\( t_{b_5} \):  parameter describing \( b_p \) [-] 
\( t_{b_6} \):  parameter describing \( b_p \) [-] 
\( t_{b_7} \):  parameter describing \( b_p \) [-] 
\( t_{br_1} \):  constant of 200 mm [mm] 
\( t_{br_2} \):  constant of 30 mm [mm] 
\( t_{br_3} \):  constant of 50 mm [mm] 
\( t_{c_1} \):  parameter describing \( c_p \) [mm] 
\( t_{c_2} \):  parameter describing \( c_p \) [mm] 
\( t_{c_3} \):  parameter describing \( c_p \) [mm] 
\( t_j \):  sand sub-base thickness [mm] 
\( t_{s_1} \):  reference sand sub-base thickness of 200 mm [mm] 
\( t_{s_2} \):  reference sand sub-base thickness of 1,000 mm [mm] 
\( U_f \):  weight assigned to a fraction [-] 
\( U_p \):  permanent vertical deformation [mm] 
\( U_r \):  resilient vertical deformation [mm] 
\( v \):  (vehicle) speed [m/s] 
\( v_{cw} \):  vertical speed of centre of gravity of carriage work [m/s] 
\( v_{max} \):  number of different vehicle/speed combinations in traffic [-] 
\( v_{uf} \):  vertical speed of the front axle including tires [m/s] 
\( v_{ur} \):  vertical speed of the rear axle including tires [m/s] 
\( v_{yf} \):  vertical speed of the bottom side of the springs representing the front tires [m/s] 
\( v_{yr} \):  vertical speed of the bottom side of the springs representing the rear tires [m/s] 
\( VV_S \):  Volders and Verhoeven sharpness (grain shape indicator) [-] 
\( VV_{S_{w}} \):  VVS of weight quantity [-] 
\( W \):  weight of material under consideration [g] 
\( W_d \):  weight of dried grains of the excluded fraction [g] 
\( W_{opt} \):  optimum moisture content [-] 
\( W_F \):  weight of fraction [g] 
\( W_u \):  under water weight of the grains of the excluded fraction [g] 
\( W_w \):  weight of wet grains of the excluded fraction [g]
\( \alpha: \) angle between \( \sigma_i \) and vertical

\( \gamma_{\text{dry, max}}: \) maximum dry density \([\text{kg/m}^3]\)

\( \gamma_{\text{gr, dry}}: \) dry density of grains in excluded fraction \([\text{kg/m}^3]\)

\( \gamma_{\text{gr, wet}}: \) wet density of grains in excluded fraction \([\text{kg/m}^3]\)

\( \gamma_{\text{s, dry}}: \) dry sample density \([\text{kg/m}^3]\)

\( \gamma_{\text{s, wet}}: \) wet sample density \([\text{kg/m}^3]\)

\( \gamma_{\text{wet}}: \) wet density \([\text{kg/m}^3]\)

\( \Delta h_{i, \lambda}: \) shift between the sine of rut depth with a wavelength \( \lambda \) introduced by the front axles of vehicles \( "i" \) and the sine of longitudinal unevenness \([\text{m}]\)

\( \Delta r_{\lambda, i}: \) shift between the sine of rut depth with a wavelength \( \lambda \) introduced by the rear axles of vehicles \( "i" \) and the sine of longitudinal unevenness \([\text{m}]\)

\( e_p: \) permanent strain \([-\text{]}\)

\( e_{p, h}: \) permanent horizontal strain \([-\text{]}\)

\( e_{p, v}: \) permanent vertical strain \([-\text{]}\)

\( e_{p, a}: \) permanent axial strain \([-\text{]}\)

\( e_{p, 2/3}: \) permanent radial strain \([-\text{]}\)

\( e_r: \) resilient strain \([-\text{]}\)

\( e_{r, ax}: \) axial resilient strain \([-\text{]}\)

\( e_{r, rad}: \) radial resilient strain \([-\text{]}\)

\( \Theta: \) sum of the principal stresses \([\text{kPa}]\)

\( \Theta_0: \) reference stress (1 kPa) \([\text{kPa}]\)

\( \lambda: \) wavelength \([\text{m}]\)

\( \lambda_{\text{max}}: \) maximum wavelength considered \([\text{m}]\)

\( \nu: \) Poisson’s ratio \([-\text{]}\)

\( \nu_{c, i}: \) Poisson’s ratio in element \( "e" \) for the following iteration \([-\text{]}\)

\( \nu_{c, \sigma}: \) Poisson’s ratio in element \( "e" \) on the basis of the calculated stresses \([-\text{]}\)

\( \nu_1: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_2: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_3: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_4: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_5: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_6: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_7: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_8: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_9: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_{10}: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \nu_{11}: \) parameter relating the Poisson’s ratio to stresses \([-\text{]}\)

\( \phi: \) angle of internal friction \([-\text{]}\)

\( \phi: \) pitch angle \([-\text{]}\)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
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<tbody>
<tr>
<td>(\phi)</td>
<td>rotational speed</td>
<td>[-/s]</td>
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<tr>
<td>(\dot{\phi})</td>
<td>rotational acceleration</td>
<td>[-/s^2]</td>
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<tr>
<td>(\phi_{cw})</td>
<td>carriage work rotational speed</td>
<td>[-/s]</td>
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<td>(\dot{\phi}_{cw})</td>
<td>carriage work rotational acceleration</td>
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<td>(\sigma_{act})</td>
<td>actuator stress</td>
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<td>(\sigma_{c})</td>
<td>wheel load contact pressure</td>
<td>[kPa]</td>
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<td>(\sigma_{cl})</td>
<td>reference wheel load contact pressure (707 kPa)</td>
<td>[kPa]</td>
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<td>(\sigma_{c1})</td>
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<td>(\sigma_{c,i})</td>
<td>wheel load contact pressure of load &quot;i&quot;</td>
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<td>(\sigma_{c,sn})</td>
<td>standard wheel load contact pressure</td>
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<td>(\sigma_{con})</td>
<td>confining pressure</td>
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<td>(\sigma_{cycl})</td>
<td>maximum dynamic actuator stress</td>
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<td>(\sigma_{dw})</td>
<td>dead weight stress</td>
<td>[kPa]</td>
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<td>(\sigma_{lw})</td>
<td>standard deviation of lateral wander</td>
<td>[mm]</td>
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<td>(\sigma_{lw-r})</td>
<td>reference standard deviation of lateral wander (200 mm)</td>
<td>[mm]</td>
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<td>(\sigma_{lw-s})</td>
<td>standardized standard deviation of lateral wander</td>
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<td>(\sigma_{rr})</td>
<td>horizontal normal stress in radial direction</td>
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<td>(\sigma_{rz})</td>
<td>vertical shear stress</td>
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<td>(\sigma_{stat})</td>
<td>static actuator stress</td>
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<td>horizontal normal stress in x-direction</td>
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<td>(\sigma_{sy})</td>
<td>horizontal normal stress in y-direction</td>
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<tr>
<td>(\sigma_{sz})</td>
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<tr>
<td>(\sigma_3)</td>
<td>third principal stress</td>
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