INTERACTION BETWEEN AN ASPHALT REVETMENT AND THE SAND SUBSOIL UNDER WAVE ATTACKS

MEDIUM SCALE EXPERIMENTS WITH OLD AND NEW ASPHALT CONCRETE

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Date: March 27, 2014
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Master of Science Thesis by
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March 2014

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Summary

Nowadays, in the Netherlands there are about 600 kilometers of asphaltic revetments lying mainly on sea dikes of which approximately 400 km of dike is covered with asphalt concrete [STOWA, 2011]. Asphalt concrete revetments are applied the most and are mostly of older age. Because of the older age it needs intensive care involving periodic inspection and safety checks. The focus on maintenance has encouraged for research to be done on asphalt revetments.

The (fatigue) behavior of asphalt concrete on a sand foundation under wave loading is not very well known. Especially about the interaction between an asphalt concrete revetment and the subsoil under wave attack only limited knowledge is available. In this study the focus is placed on getting more detailed insight into the interaction between an asphalt concrete revetment and the underlying subsoil (foundation) under wave loading for both old and newly made asphalt concrete.

Deltares and KOAC-NPC have been focusing on the interaction between an asphalt revetment and the subsoil under wave attack and have been performing experiments together with finite element calculations to predict and validate their data. Medium scale experiments which simulate a loading caused by wave attack on a scaled part (about a factor 3) of an asphalt concrete dike revetment were designed. A series of short pulse (haversine) loadings were applied to simulate the stresses and strains during wave attack.

In total six medium scale experiments (4 on newly made and 2 on old asphalt concrete) have been carried out and after each of the experiments the test set-up and/or procedure had been improved.

The medium scale experiments almost resemble a falling weight deflection measurement. With the deflections measured during the experiments the occurring strains can be calculated. Using the WESLEA software the elastic deflection profile for a selection of pulse loadings measured during the experiments are back-calculated for selected measurement points of experiments 3, 4, 5 and 6.

The linear elastic calculations with the WESLEA software led to a better understanding of the behavior of the asphalt concrete plate on the Baskarp sand for both the old and new asphalt concrete.

Back-calculations for all of the loading stages during experiment 5 on old asphalt concrete with the ABAQUS model used by Deltares and KOAC-NPC have been performed, using a linear elastic material model for both the asphalt concrete plate and the Baskarp sand bed. The fits are reasonable, but a low stiffness modulus is found.

From the back-calculations of the medium scale experiments with WESLEA, typical stiffness moduli for the Baskarp sand bed (varying between 40 to 60 MPa) are found, depending on
the loading conditions. These values are also sufficient for the linear elastic calculations with ABAQUS model for experiment 5.

A constant stiffness modulus of the asphalt concrete could not be determined. It is observed that the stiffness modulus is different for each load magnitude, even for the same asphalt concrete plate on the same Baskarp sand bed. The stiffness modulus of the asphalt concrete plate depends on the strain level and the loading speed [KOAC-NPC, 2013]. The uncertainty of the asphalt concrete stiffness modulus also depends on the varying thickness of the asphalt concrete plate and the accuracy of the measured elastic deformations.

Since most of the current asphalt concrete revetments in the Netherlands consist of aged asphalt concrete (about 90% is older than 20 years), the failure mechanism most likely to occur under wave impacts is failure due to fatigue. From the research done up till now, it has become clear that if the asphalt concrete revetments which are less affected by ageing are less vulnerable to failure due to fatigue under wave impacts.

**Keywords**: fatigue, interaction between asphalt concrete and sand, asphalt concrete revetment, medium scale experiments, wave impact, Baskarp sand, deflection profile
Preface

As a student with the specialization Hydraulic Structures one of the compulsory parts during the second year of the Master program in Hydraulic Engineering is that a Master thesis needs to be written. In doing this it is opted for a research in the field of asphalt concrete revetments, which is treated in this report.

Deltares and KOAC-NPC have been conducting research on asphalt concrete dike revetments in particular investigating failure due to wave impacts. In this data from the research on asphalt concrete revetments as performed by Deltares is used for this study to gain more insight into the interaction between asphalt concrete revetment and a (Baskarp) sand foundation.

With this report I hope to contribute to a better understanding of the interaction of asphalt on the subsoil.

I would like to express my deepest appreciation to all those who provided me the possibility to complete this report. A special gratitude I give to my graduation committee, whose contribution in stimulating suggestions and encouragement, helped me to coordinate my thesis project especially in writing this report. I would like to thank Ir. Rien Davidse for helping me understand the calculations done with the ABAQUS software. I am very thankful to Deltares and TU Delft, especially Dr. B.G.H.M. Wichman and Ir. L.J.M. Houben, for making available the facilities, the knowledge and support while writing my thesis.

Sincerely,

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List of symbols and abbreviations

α = Slope angle [°]
α_T = Transition Surf Rad
a = Regression coefficient [-]
b = Regression coefficient [-]
\( \beta_{dp} \) = Internal friction for the Drucker-Prager material model [°]
c = Modulus of subgrade reaction [N/m^3]
C_C = Compression index [-]
C_s = Expansion index [-]
°C = Degrees Celcius
E_{mix} = Young’s modulus [MPa]
\( \epsilon \) = Maximum strain at the bottom of the asphalt layer [μm/m]
\( \epsilon_{t,T} \) = Strain as a function of loading time t and temperature T [m/m]
\( \epsilon_{pl.0} \) = Initial Yield Surf Pos
ξ = Iribarren number [-]
f = frequency [Hz]
g = Gravitational acceleration [m/s^2]
H = Wave height [m]
H_s = Significant wave height [m]
H = Thickness of asphalt layer [m]
Hz = Hertz
K = Coefficient for Drucker Prager model [-]
K = Compression modulus [-]
km = Kilometers
kN = KiloNewton
L_0 = Wave length [m]
m = Meters
min = Minutes
mm = Millimeters
MPa = MegaPascal
N = Number of loadings [-]
N_{f,i} = Number of loads to failure at stress level I [-]
\( n_i \) = Number of loads at stress level i [-]
\( p_{first} \) = First loadings [kN]
\( p_{last} \) = Last loadings [kN]
\( p_{max} \) = Maximum wave impact pressure [N/m^2]
\( p_{t,el} \) = Tensile limit [MPa]
\( p_{aggregate} \) = Specific gravity aggregate [kg/m^3]
\( p_{bitumen} \) = Specific gravity bitumen [kg/m^3]
\( p_{mix} \) = Specific gravity of compacted asphalt mix [kg/m^3]
\( \rho_w \) = Density of water [kg/m^3]
q = Factor of impact based on experiments on different slopes
\( \Phi \) = Angle of internal friction [°]
R = Cap eccentricity
S = Stiffness modulus of asphalt [N/m^2]
S_{mix} = Stiffness modulus of asphalt concrete mix [MPa]
S_{bit} = Stiffness modulus of the bitumen (MPa)
s = Seconds
σ = Bending stress in asphalt layer [N/m²]
σ₀ = Yield stress [MPa]
σ₀ = Applied bending stress [MPa]
σ₀,5% = Yield stress with a 5% chance of exceeding [MPa]
t = Time [s; min]
ΔT = Difference in temperature [°C]
μm = Micrometer
Vₐ = Volume percentage of air [%]
V₉ = Volume percentage of bitumen [%]
V₈ = Volume percentage of aggregate [%]
ν = Poisson’s ratio of asphalt [-]
z = 0.5 x width of the load [m]
% = Percentage
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Chapter 1 Introduction and scope of this study

In this first chapter some of the general background of this study is introduced. The use of asphalt revetments and the research which has been conducted on asphalt revetments are treated, which leads to the focus of this study.

1.1 Asphalt revetments

A (dike) revetment is a structure which protects a slope of a dike body of soil or other materials against erosion. In most cases it is the only protection the dike has against wave impacts or currents. There are different materials which can be used for a revetment. One of these materials is asphalt.

The use of asphalt or bitumen in waterworks is a practice which started thousands of years ago. In the ancient Middle East, the local residents of the area between the river Euphrates and the river Tigris, called Mesopotamia, used natural asphalt to glue stones together in revetments along the shores. In Mesopotamia natural asphalt was also used for making temple baths and water tanks watertight. Nowadays some of these structures built many years before are still intact and therefore provide us proof of the high durability of asphalt as a material [VBW, 1984].

![Figure 1: Revetment along the banks of the Tigris [VBW, 1984]](image)

The first applications of asphalt in the Netherlands date from the 1920s and 1930s (Aarkanaal, Julianakanaal) as factories that produce asphalt (bitumen) artificially as product of the petroleum industry entered the Dutch hydraulic engineering. Especially after the North Sea flood of 1953 the use of asphalt in revetments along the coast of the Netherlands largely increased [VBW, 1984]. After this disaster a large number of dikes had to be restored as soon as possible, while a sufficient quantity of clay and rock material was not present at that time. Asphalt was used as a dike revetment material, because the application of asphalt could be a quicker and less labour-intensive process than the application of conventional embankment materials. Asphalt could also provide a better waterproof layer, limiting loss of stability of the inner slope due to penetration of water through the dike. Asphalt could be placed much faster than the materials usually applied at that time, such as basalt columns. The experience
gained from the reconstruction activities after the flood were such that it was decided to also apply asphalt revetments on a large scale for the Delta Works.

Asphalt concrete is used as a dike revetment to protect the dike body from erosion. In general the subsoil is sandy and the thickness of the asphalt layer varies between 150 and 400 mm.

Figure 2: Typical cross-section of a dike with an asphalt revetment [www.ecomare.nl]

Roughly 30 percent of the Netherlands lies below sea level, which creates the necessity for flood defenses. Nowadays, in the Netherlands there are about 600 kilometers of asphaltic revetments lying mainly on sea dikes and can be distinguished into 3 types [STOWA, 2011]:

- Asphalt concrete: approximately 400 km of dike is covered by this type of revetment, of which 90% is older than 20 years and 70% older than 30 years.
- Open stone asphalt: for about 100 km of dike line.
- Asphalt penetrated crushed stones: around 100 km of dikes are covered by this type.

Figure 3 illustrates the differences in composition for each type of asphalt revetments.

Figure 3: Composition (in % by volume) of the different types of asphalt concrete [TRB Asfalt, 2012]

Asphalt penetrated crushed stones and asphalt concrete are mainly used because of their high strength and durability. The focus of this report will be on asphalt concrete, which has been applied the most and is mostly of older age (which affects the bitumen properties).
1.2 Research on asphaltic dike revetments

In the previous paragraph, the use of asphalt as a material for Dutch dike revetments which have been designed and built in the past is presented. In this paragraph an important aspect for nowadays, regarding the service life of a (dike) revetment, namely maintenance, is treated.

Up till now most of the asphalt concrete revetments constructed in the past 30 to 35 years have functioned very well without failure. However, for about 100 km of the asphalt revetments had to be reconstructed, because of the loss of strength and/or damage. All aged asphaltic concrete needs intensive care involving periodic inspection and safety checks. The focus on maintenance has encouraged for research to be done on asphalt revetments. That is why in 2007 Rijkswaterstaat and STOWA (StichtingToegepast Onderzoek Waterbeheer) have initiated extensive research on safety, inspection and management of especially old (existing) asphalt concrete revetments. This research is of great importance since the asphalt concrete revetment often is the only protection against erosion for dikes which keep large parts of the Netherlands dry. Dikes in the Netherlands are in most cases structures which separate the dry land from the sea or river. This is why experts of Deltares, KOAC-NPC and TU Delft have done/are doing their parts within this research program.

Through the years inspection has been carried out on the asphalt revetments from which it can be learned that old asphalt revetments are more sensitive to failure (due to aging and intrusion of water) than asphalt revetments that have just been constructed [Land+Water nr.6/7, 2009]. For example, in 2006 the asphalt revetment on the Waddenzeedijk in Friesland has shown signs of stripping, even after the dike had been approved to be safe enough and in a good state in 2004.

Figure 4: Inspection of an asphalt revetment in Friesland (Westhoek-Zwarte Haan in 2006) [Land+Water nr.6/7, 2009]
Another common type of failure of an asphalt revetment is fatigue because of the repeated loading by wave attack under (moderate) storm conditions. The effects due to fatigue and deformation of the subsoil are illustrated in Figure 5, where the signs of failure can be observed for this failure type. In paragraph 2.3 the different types of failure mechanisms will be treated in more detail.

Figure 5: Failure due to deformations (fatigue) [12th Baltic Sea Geotechnical Conference 2012]
1.3 Problem description

There are different types of failure for asphalt revetments, which lead to different research topics. One of these topics is to investigate the fatigue behavior of asphalt, especially in the case of a revetment. Deltares and KOAC-NPC have been focusing on the interaction between an asphalt revetment and the subsoil under wave attack and have been performing experiments together with finite element calculations to predict and validate their data.

Experiments have been performed to determine the necessary asphalt properties and medium scale tests were set up which simulate the in situ situation of an asphalt concrete revetment on a sandy dike. Finite element calculations show that the values of the asphalt properties obtained need to be adjusted in order to simulate the deflection measurements of the medium scale tests. The stiffness for asphalt concrete is a factor 2 to 3 smaller for the model of the finite element calculations than what was determined with the four-point bending tests [KOAC-NPC, 2013]. This is the case for both the new (laboratory) and the old (in situ) asphalt concrete used for the experiments conducted by Deltares. The medium scale experiments will be treated in more detail in chapter 3.

A real (large) scale experiment is under consideration, in which periodic loading by wave attack on an asphalt concrete revetment supported by sand will be simulated. The test results can be compared with smaller scale tests and fitted with finite element calculations. Finite element calculations can also be used to predict the loading conditions for this real scale experiment. It is therefore important to understand the model and the input parameters. This has to be investigated with the available literature and data from the (medium scale) laboratory experiments performed.

The (fatigue) behavior of asphalt concrete on a sand foundation is not very well known. Especially about the interaction between an asphalt concrete revetment and the subsoil under wave attack only limited knowledge is available.
1.4 Objective
In this study the focus will be placed on getting more detailed insight into the interaction between an asphalt concrete revetment and the underlying subsoil (foundation) under wave loading for both old and newly made asphalt concrete.

The method of research in this study is formulated as follows:

1. A literature study will be performed to get to know more about asphalt concrete as a revetment under wave attack with the focus on the behavior of asphalt concrete under wave loading conditions.
2. The medium scale experiments and finite element calculations conducted by Deltares and KOAC-NPC will be considered to get a better understanding of what is already known and which data is available or still needs to be determined.
3. By back-calculating the measured elastic deflections from the experiments an attempt is made to get insights into the interaction between asphalt and sand underneath. Also differences between old and new asphalt can be discovered using this approach.
4. Seek for improvements for the calculations with the existing models in ABAQUS which will also improve the predictions.

1.5 Readers guide
The structure of this report roughly follows the above formulated research method. In this first chapter an introduction is given for the study which is discussed in this report. Followed by chapter 2 where the important information from the literature study is provided. In chapter 3 the conditions and improvements of the experiments performed by Deltares are explained. In chapter 4 the data from the medium scale tests have been used for back-calculations with WESLEA software. In chapter 5 the measured deflections have been fitted with the ABAQUS software to improve the existing model. In chapter 6 the conclusions and recommendations are stated.
Chapter 2 Literature study
It is important to know about asphalt and asphalt revetments under wave loading. In this chapter the aspects of this material regarding the properties and the behavior will be given. In the first two paragraphs information about the structure and properties of asphalt are presented. In the third paragraph the known failure mechanisms are mentioned. In the fourth paragraph the types of waves and design conditions are treated. And in the fifth paragraph the GOLFKLAP program is introduced.

2.1 Asphalt and bitumen
Asphalt concrete is the name for a mix of certain materials. It consists of mineral aggregate (sand, filler, gravel or crushed stones) and bitumen. Bitumen is a visco-elastic material, which causes an asphalt concrete mixture to be visco-elastic as well. This means that asphalt can deform (plastic) under a long duration of loading, while for loadings with a short duration it behaves as an elastic solid material.

Asphalt (bitumen) can be found in natural deposits in lakes such as the Pitch Lake in Trinidad and Tobago and Lake Bermudez in Venezuela and also along the coast of the Dead Sea and in asphalt/bitumen-impregnated sandstones known as bituminous rock and "tar sands".

Bitumen may also be a refined industrial product, which can be subtracted from crude oil by e.g. fractional distillation (usually under vacuum conditions). A better separation can be achieved by further processing of the heavier fractions of the crude oil in a de-asphalting unit, which uses either propane or butane in a supercritical phase to dissolve the lighter molecules which are then separated. Further processing is possible by "blowing" the product, namely, by reacting it with oxygen. This makes the product harder and more viscous. Asphalt concrete can be produced by heating and mixing the bitumen with heated and dried aggregates at a temperature of 150-180 °C. Another way to produce asphalt concrete is to add a bitumen emulsion to the aggregates (emulsion is used to prevent cluttering of the bitumen).
The primary use of bitumen is in road constructions, where it is used as the glue or binder mixed with aggregate particles to create asphalt concrete. Its other main uses are for bituminous waterproofing products, including production of roofing felt and for sealing flat roofs.

Years of experience and gained knowledge in the Netherlands have helped to develop the best techniques and materials for application of asphalt. The diversity in mixture compositions and properties, the different methods of application (prefabricated mats or in situ asphalting) and environmental safety aspects allow for asphalt to be a material which is very suitable for application in both large and small hydraulic works. Asphalt as a material in hydraulic structures can fulfil a water retaining and erosion limiting function on banks in combination with the conservation of the local landscape, natural history and cultural values.

An asphalt concrete revetment acts as a plate, therefore the required layer thickness is considerably less than that of element revetments. The most used type of bitumen in hydraulic engineering is 70/100 (in the past it was 80/100) [TRB Asfalt, 2012]. This is a measure for the penetration (expressed as 0.1 mm) of a needle in the bitumen in a standardized test at 25 °C. The higher the penetration, the softer the bitumen. The preferred flexibility of the revetment in the Netherlands is such that the 80/100 bitumen would be adequate [TRB Asfalt, 2012].

The strength of an asphalt mixture is partly determined by:

- The dimensions and shape of the mineral aggregate, which provides a degree of rigidity to the material.
- The quantity (ratio) of the mineral aggregate with respect to the percentage of bitumen present in the mix.
- The voids in the mixture, which can be filled with bitumen in order to produce watertight asphalt concrete mixtures.

There are many possible applications for asphalt in waterworks besides application as dike revetments, such as (see figures 8 to 11):

- Toe protections.

*Figure 8: Toe protection of penetrated asphalt – Veersedam- 1991 [TRB Asfalt, 2012]*

- Dunes (these are more land inward than relative to the position of revetments).
In order to be used as a dike revetment, the asphalt needs to be strong enough to withstand the occurring hydraulic loads and flexible enough to overcome differential settlements of the dike. In addition the stability of the asphalt concrete needs to be sufficient for application on slopes of dikes. Asphalt concrete, asphalt penetrated crushed stones and open stone asphalt meet these criteria and are suitable for application as a revetment [TRB Asfalt, 2012].
2.2 Asphalt properties

Since asphalt is a mixture, the properties depend on the type, quality and the composition of the mineral aggregates and bitumen. The properties of asphalt are important factors to be considered if to be used as a material for a revetment. If certain properties are desired it is essential to choose the right composition and suitable materials.

The most important aspects which are considered for asphalt concrete are given as follows [TRB Asfalt, 2012]:

- **Permeability**: The amount of holes and how well they are connected with each other are a measure for permeability. Most revetments need at least to be sand tight but most of the times also watertight. Mixes with a low void content (e.g. 3%) or applying a surface treatment layer makes the asphalt concrete hardly accessible to external factors. The more an asphalt revetment is accessible to external factors, especially in case of a high void contents, the more the adhesion between bitumen and mineral aggregate becomes of importance.

- **Mechanical properties**: for hydraulic structures both temperature and loading time do vary a lot, especially along the coast. In the Netherlands the temperature of an asphalt revetment may vary from several degrees Celsius below zero till fifty degrees above zero. The loading time can vary from 0.1 second (peak loads due to wave impacts) to several years (settlement of the surface). Asphalt is stiff and strong when short loaded and at low temperatures, but at high temperatures and with long loading times it behaves as a viscous material thus flexible and weak. The mechanical properties which are very important for this material are:
  - **Stiffness**: the resistance to loads which is defined as a quotient between applied stress and resulting strains. The stiffness is also dependent on the amount of voids in the material and the amount and type of bitumen. Because of the visco-elastic properties of the bitumen, also an asphalt mixture exhibits visco-elastic behavior, which means that the stiffness depends on temperature and loading time.
  - **Strength**: a measure for the maximum stress which the asphalt can resist. In case of repeated wave loading especially flexural strength and the fatigue behavior under flexural tensile stress are very important parameters in designing an asphalt revetment.
  - **Stability**: the ability to prevent permanent deformation under the influence of a constant load (long loading times). This has proven not to be a problem in Dutch practice when applied on slopes, which are not too steep.
  - **Flexibility**: the ability to be able to undergo deformations in which the asphalt concrete remains intact. Only when large deformations occur rapidly, the asphalt layer cannot follow the deformation.
• **Durability:** this indicates for how long the relevant properties will remain on the safe side (no failure). Some asphalt revetments in the Netherlands are almost 60 years old, therefore checking this property is an interesting case. The properties of asphalt are changing in time due to the exposure to oxygen, water and ultraviolet light. The most important ageing mechanisms influencing the strength in time are [TRB Asfalt, 2012]:

  o **Stripping:** the separation of the bitumen from the aggregate surfaces (loss of adhesion) due to the intrusion of moisture into the asphalt concrete. This reduces the flexural tensile strength of the asphalt concrete. A low amount of voids and surface treatment slows down the process of stripping. A salty environment can accelerate the process of stripping.

  o **Hardening of bitumen:** exposure to light and atmosphere causes hardening of the bitumen. Hardening of the bitumen causes the material to become more brittle; the flexural tensile strength increases and the strain at break decreases. As a result the material becomes more sensitive for cracking. Cracks in aged asphalt concrete are often temperature cracks, caused by hardening of the bitumen. Higher temperatures and larger voids contents lead to a higher rate of hardening.

  o **Fatigue:** Asphalt is sensitive to fatigue, which means that the previously mentioned mechanical properties (especially strength) decrease as the material is loaded more frequently. This aspect is only relevant for stiff mixtures constantly loaded. Asphalt revetments are temporarily repeatedly loaded in practice only under extreme (storm) conditions. During the summer, the effect of fatigue largely disappears again, because higher temperatures have a healing effect on asphalt, the cracks caused by fatigue close again. As the asphalt ages and becomes more brittle, fatigue will cause a lasting change on which healing will have less effect.

  o **Erosion:** a water flow can have an eroding effect on asphalt mixtures, especially when the water carries solid particles. The collision forces of these particles with the asphalt surface cause stresses in the asphalt concrete which will increase with greater hardness of the binder. It follows that the lower the minimum temperature at which erosion may occur, the higher penetration value of the bitumen should be.

  o **Biological infestations:** damage to the revetment caused by algae, barnacles, roots of a plant, etc. The viscous behavior of asphalt enables organisms to gradually deform the material. Mostly the surface of the asphalt can be treated to prevent this from emerging.

  o **Chemical deterioration:** Bituminous materials are chemically inert, except against some hydrocarbons. However the concentration of the hydrocarbons
must be very high if damage is to occur. A surface treatment with bitumen can prevent damage.

The lifetime of an asphalt mixture applied as a revetment on a dike body mainly depends on internal and external factors, namely the:

- Durability of the adhesiveness between mineral aggregate and bitumen
- Influence of external factors such as moisture, temperature, vegetation, etc.
- Presence of floating debris and light rubble that will roll over the coating and cause especially in open stone asphalt damage
- The presence of regular heavy wave attack

As mentioned before, the adhesiveness of the asphalt mixture plays an important role in keeping together the asphalt plate as a revetment. The adhesion between the mineral aggregate and bitumen is determined by the given features [TRB Asfalt, 2012]:

- Amount of bitumen in the asphalt concrete mix (especially relevant for stripping and ageing)
- Properties of the bitumen
- The fillers added to the asphalt concrete mix
- Filler-bitumen ratio, which determines the viscosity of the mortar/mix
- Use of calcium hydroxide and adhesion promoters
- Properties of the mineral aggregate in the asphalt concrete mix (also the roughness)

Rupture in asphalt mixtures occurs when a certain (tensile) stress is exceeded. Because asphalt mixtures display fatigue behavior, the allowable stress decreases as the material is loaded more frequently. The number of wave impacts is therefore an important input data when determining the lifetime of an asphalt concrete revetment. The allowable stress is determined in a relationship between the applied stress and the number of load repetitions. This relationship is the fatigue curve and can be determined by performing laboratory tests or with the help of nomograms. For asphalt concrete revetment the fatigue curve is taken to be a nonlinear curve. In Figure 12 an example is given for such a fatigue curve.

![Figure 12: Example of a fatigue curve for an asphalt concrete revetment [TRB Asfalt, 2012]](image)
For determining the non-linear fatigue curve for in situ asphalt concrete revetments cores are drilled out of the revetment, from which two test pieces are sawed per core. For one specimen the yield stress is determined. The other specimen undergoes a fatigue test. With this being done enough data is generated for determining the fatigue curve.

The fatigue relationship in general form is stated as [TRB Asfalt, 2012]:

$$\log(\log(N)) = \beta + \alpha \log(\log(\sigma_b) - \log(\sigma_0))$$

In which:
- $N$= number of loadings [-]
- $\sigma_b$= yield stress [MPa]
- $\sigma_0$= applied bending stress [MPa]

Using a linear regression on double log scale, the coefficients $\alpha$ and $\beta$ are determined. This regression provides an estimate of the expected value for the fatigue behavior. In assessing an asphalt revetment some security should be incorporated. This is done by making use of the uncertainty in the flexural breaking strength. The fatigue curve to be used in the assessment is in fact given by [TRB Asfalt, 2012]:

$$\log(N) = \beta(\log(\sigma_{b,5\%}) - \log(\sigma_0))^{\alpha}$$

In which:
- $\sigma_{b,5\%}$= the 5% lower boundary of the respective flexural strength [MPa]

Important factors considered for the difference between old and new asphalt are listed:
- Penetration Percentage of bitumen
- Flexural strength (decreases in time)
- Percentage of voids
- Amount of stripping (in case of old in situ asphalt)
2.3 Failure mechanisms for asphalt revetments

The failure mechanisms of asphalt concrete revetments are physical phenomena for which the revetment under loading undergoes an intolerable deformation so that the consistency of the construction is lost. This may cause the asphalt revetment to no longer provide protection against erosion of the dike body. In that case it could lead to failure of the dike together with the consequences of such.

A list of the hydraulic loads which could be expected on revetments is given:

- Water pressure
- Uplift
- Air pressure
- Wave attack
- Current
- Settlement of the subsoil and/or dike body

Other (special) loads that could be acting on an asphalt revetment are:

- Traffic, both during the construction and the in-service phase
- Vandalism, whether or not the result of recreational activities
- Floating waste on the revetment
- Collisions and anchorage
- Ice on the revetment

These loads can lead to mainly 3 types of failure mechanisms for asphalt concrete revetments, namely [TRB Asfalt, 2012]:

1. **Failure caused by wave impact**: A wave that collapses on the slope causes an impact pressure on the revetment. Repeatedly loading like such means that beside flexural strength also fatigue will play a role here.

[Figure 13: Sketch of wave impact load on a revetment [KOAC-NPC, 2009]]
The revetment must be sufficiently resistant against this impact and should not fail due to the wave loading. This failure mechanism can occur in one or more of the following mechanisms:

a. **Failure by exceeding the flexural strength**

![Figure 14: Failure by exceeding the flexural strength [TRB Asfalt, 2012]](image)

b. **Failure by exceeding shear strength**: This is especially a problem when the constructed layer is thin (< 10 cm).

![Figure 15: Failure by exceeding the shear strength [TRB Asfalt, 2012]](image)

c. **Failure of the subsoil**: In case of heavy wave loading the occurring mechanisms can lead to failure of the subsoil and therefore the asphalt revetment and therefore the dike.

Situations that may take place and make the chance of failure larger are:

- Over saturated subsoil subjected to wave loading (high phreatic level in the dike body)
- Sliding of the revetment
- Formation of an S-shaped surface profile
- Failure of the subsoil (bearing capacity exceeded)
2. **Water overpressure causing uplift of the revetment**: In case of an impermeable revetment water pressures can occur below the revetment. In the event of a high outside water level, the phreatic line in the dike body will rise because of the head difference between the outer water and groundwater in the dike body. In most cases an extreme water level can be followed by a rapid fall of the outside water level. This means that the phreatic line inside the dike body could be higher than the outside water level. This could cause failure if the uplift pressure exceeds the counter pressure by the weight of the asphalt layer.

![Diagram](image1.png)

*Figure 16: Failure by exceeding the shear strength [TRB Asfalt, 2012]*

3. **Material transport in case of open stone asphalt**: The asphalt revetment must prevent material from the dike body to erode. During service an asphalt revetment may get damaged due to different causes that may lead to material transport from the subsoil (erosion). The causes may be:
   - Due to aging cracks can be formed much easier
   - Intrusion of water
   - Special load cases: collision, ice, heavy equipment, etc.
   - Design flaws

Besides these failure mechanisms there are also 2 related to failure of the toe protection:

1. Overpressure caused by wave motion
2. Erosion of the foreshore

For this study the focus is on asphalt concrete as a revetment with the focus on the failure mechanism caused by wave impact. For this mechanism 3 possibilities are known of which failure caused by exceeding the flexural strength of the asphalt concrete (failure mechanism 1a) is expected to occur. However, it is kept in mind that it is possible that failure 1b or 1c can also take place.
2.4 Wave impact
One of the most important loads to be considered on a revetment is the load caused by wave action. Especially since the focus of this study is on the failure mechanism caused by wave attack. In the following paragraphs the breaker types and design conditions of (breaking) waves will be treated.

2.4.1 Breaking waves
A wave breaks as it approaches a revetment (shoreline), because the wave becomes too steep and/or if the water becomes very shallow. For this study it is assumed that the waves working on a revetment are breaking waves. Different types of breaking waves depend on the wave steepness and slope of the revetment and can be distinguished with the dimensionless Iribarren number or surf similarity parameter. The parameter is defined as [Schiereck G.J., 2004]:

$$\xi = \frac{\tan \alpha}{\sqrt{H/L_0}}$$

In which:  
\(\xi\) = Iribarren number [-]  
\(\alpha\) = angle of the slope of the revetment [°]  
\(H\) = wave height [m]  
\(L_0\) = wave length [m]  

\(\xi\) represents a ratio between the slope steepness of the revetment (tan\(\alpha\)) and the steepness of the occurring wave (\(\sqrt{H/L_0}\)). Different breaker types with their characteristic Iribarren number are given in Figure 17.

![Figure 17: Breaker types for some waves](image-url)
The plunging and collapsing breaker behavior of waves give the highest impact pressures as they strike the slope (revetment) more severely with every loading than that surging and spilling breakers do. For this reason only the effects of plunging and collapsing breaker types are considered in this report.

2.4.2 Design conditions
Asphalt revetments are often located in the wave attack zone of a dike. The main propose is to protect the sandy or clay dike body against wave attacks. Asphalt revetments in the Netherlands are usually designed to withstand a storm which may happen every 4000 or 10000 years. The significant wave height is taken up to 4.5 m and the duration of the design storm is taken to be 35 to 45 hours [De Loof A.K. et. al., 2004]. During this period, the revetment is severely and cyclically loaded by wave attack. The position of the impact point is defined as the location of the highest impact pressure that occurs per wave attack. Due to the so-called wind setup, both the low and high tide levels increase. In the Netherlands a typical value for the wind setup is 3.50 m [De Loof A.K. et. al., 2004].

The waves of the North Sea between England and the Netherlands have a typical period of 5 to 10 seconds. However, the impact period of a breaking wave on the dike revetment has a much shorter duration up to 0.3 s. In the present analyses, a wave period of 10 s is assumed and the wave attack duration is taken to be 0.1 s. This value is within the range observed in the Delta flume tests on asphalt in 1991.
2.5 GOLFKLAP software

For the design of an asphalt dike revetment mainly design graphs with distinct lines for the layer thickness depending on the significant wave height are used [TRB Asfalt, 2012]. For special cases (parts of an) asphalt revetments can be designed and analyzed on wave impacts with the help of the computer software: GOLFKLAP. Also for the safety assessment of older asphalt concrete revetments the GOLFKLAP software can be used. The asphalt revetment as a layer can be considered as an elastic plate supported by a spring (subgrade) foundation (Winkler spring model). The system is subjected to bending due to wave impact on the asphalt plate. The wave impact load is schematized as a series of triangular loads (see Figure 18) [De Loof A.K. et. al., 2004].

![Figure 18: Schematization of GOLFKLAP model [12th Baltic Sea Geotechnical Conference 2012]](image)

The GOLFKLAP software requires adequate input parameters for the material properties. The input parameters will be treated together with the explanation of the model. According to the GOLFKLAP model the stresses are calculated with the following formula [De Loof A.K. et. al., 2004]:

\[
\sigma = \frac{p_{\text{max}}}{4\beta^3 Z} \left(1 - e^{-\beta z (\cos(\beta z) + \sin(\beta z))}\right) \frac{6}{h^2}
\]

With: \( \beta = \sqrt{\frac{3c(1-\nu^2)}{Sh^2}} \)

In which:  
- \( \sigma \) = bending stress in asphalt layer \([N/m^2]\)
- \( p_{\text{max}} \) = maximum wave impact pressure \([N/m^2]\)
- \( z = 0.5 \times \text{width of the load} \) [m]
- \( h = \text{thickness of asphalt layer} \) [m]
- \( c = \text{modulus of subgrade reaction} \) [N/m³]
- \( S = \text{stiffness modulus of asphalt} \) [N/m²]
- \( \nu = \text{Poisson’s ratio of asphalt} \) [-]
The maximum wave impact pressure can be calculated taking into account the slope of the revetment with the following formula [De Loof A.K. et. al., 2004]:

\[ p_{\text{max}} = \frac{0.25}{\tan(\alpha)} \times \rho_w \times g \times q \times H_s \]

In which:
- \( \alpha \) = slope angle [°]
- \( \rho_w \) = density of water [kg/m³]
- \( g \) = gravitational acceleration [m/s²]
- \( q \) = factor of impact based on experiments on different slopes
- \( H_s \) = significant wave height [m]

The software calculates the (tensile) stress at the bottom of the asphalt layer for an array of points along the slope of a dike for every wave loading during a storm. Every wave load adds damage to the construction. With the Miners rule it can be evaluated whether the revetment can resist the occurring sum of wave loads. In formula [De Loof A.K. et. al., 2004]:

\[ \sum \frac{n_i}{N_{f,i}} \]

In which:
- \( n_i \) = number of loads at stress level i [-]
- \( N_{f,i} \) = number of loads to failure at stress level i [-]

The revetment will fail when the Miner sum reaches or exceeds 1.
Chapter 3 Medium scale tests

In this chapter the experimental research performed by Deltares, in particular the medium scale experiments, is explained. The data generated from this research forms an important basis for this study. The aim, test set-up and research method of the medium scale experiments is summarized.

3.1 Aim of the experimental research

Nowadays, for sea dikes in the Netherlands the interaction between an asphalt revetment on a dike body of sand under wave attack is unknown for a large part. This is the reason why Deltares is doing research in this field. The aim of this research is to validate the calculation method of the GOLFKLAP software program and to improve it, with the main focus on the interaction of the asphalt and the underlying sand bed.

A real (large) scale test, performed at first in the laboratory, can give more certainty about the quality of asphalt revetments and thus will give more certainty in determining the point of failure. By using this insight, the GOLFKLAP model can be improved in order to make a better and more accurate safety assessment of the existing asphalt revetments on dikes.

In order to do this at first medium scale experiments were performed. The goal of the medium scale tests was to investigate the effect of the bearing capacity of the subsoil on the fatigue behavior of the asphalt concrete and to improve the existing calculation method. Using the experience gained from the medium scale experiments, a large scale experiment has been designed. With the improved calculation method the design and testing procedure of the large scale experiment can be optimized. By performing a series of medium scale experiments, more insight will be gained into the (mechanical) behavior of the important asphalt and subsoil parameters in a controlled environment. A total of 6 medium scale tests have been performed (up till now).

The medium scale experiments set-up simulates a loading caused by wave attack at a part of the dike body on a smaller scale (see Figure 19). The applied load in this experiment (looking at the pressure distribution) is not similar to that of a wave attack. However, the loading conditions (with a cylindrical loading head in the middle of the specimen) have been chosen such that the stresses and strains in the asphalt directly under the load are similar to those expected in the field when performing falling weight deflection measurements. In addition, the thickness of the asphalt layer and the testing conditions were taken such that the shear strength of the asphalt plate would not be exceeded (see mechanism 1b in section 2.3).
Important research questions which are investigated by performing the medium scale tests are [Deltas, 2012]:

- Does the relative density of the sand body change under repeated loading?
- Does the stiffness of the asphalt decrease during repeated loading?
- Is the sand layer able to carry a larger part of the load after repeated loading?
- How is it possible to measure the failure of the asphalt plate?

For most of the research questions careful answers are given, but more research needs to be done to have more certainty of these answers.
3.2 Test set-up and preparations
The medium scale test set-up simulates wave attack at a scaled part (about a factor 3) of the dike body. The test set-up is built up of four steel rings which form a cylinder with a total height of 1m and a diameter of 0.9 m (see figure 20).

![Figure 20: The medium scale laboratory test set-up with the loading head (left) [Deltares, 2012]](image)

The cylinder is filled with 0.8 m of Baskarp sand which represents the subsoil and a circular asphaltic plate of approximately 50 mm thickness on top of it. In the center of the asphalt plate a cylindrical loading head was mounted. For all experiments a rubber plate of 3 mm thickness was placed between the asphalt plate and the loading head to attain a uniform stress distribution applied on the asphalt plate. Also 6 transducers were mounted onto a frame above the asphalt plate to measure the deflections during the experiments.

A relative density of the sand of about 65% was intended, typical for a medium dense sand bed directly under an asphalt revetment as found in practice. This value around 65% was aimed for during the medium scale tests, except for experiment 1 where the relative density was around 85%.

At the center of the circular asphalt plate the loading is applied by using the cylindrical loading head. A series of short pulse (haversine) loadings were applied to simulate the stresses and strains during wave attack. The pulse loadings are driven hydraulically. The loading head was pressed on the asphalt plate (a relatively small constant offset load was used) to prevent the plate to move upwards and to assure that the loading head remains in contact with the asphalt plate.
The pulse loading of the loading head increases and decreases with a haversine shape (see figure 21).

![Figure 21: Pulse loading simulating wave attack [Deltaires, 2012]](image)

In total six medium scale experiments have been carried out and after each of the experiments the test set-up and/or procedure had been improved. Adjustments in between the experiments are (see also Table 1):

- After carrying out experiment 1, the asphalt mix was adjusted for experiments 2, 3 and 4. The asphalt concrete for experiment 1 was coarser which caused inaccuracy because of the niches and also the grains were rather large compared to the asphalt plate thickness.
- The footprint of the loading head for tests 1 and 2 was ø50 mm and for tests 3, 4, 5 and 6 it was ø100 mm. This together with a 3 mm thick rubber plate ensures for a better spreading of the load and prevents the loading head from sinking into the asphalt plate.
- The temperature was held constant at 20 °C during the first two experiments and at about 10 °C during experiments 3, 4, 5 and 6. This decision was made to avoid unwanted deformations under the loading head.
- The loading forces and durations also differ for each experiment. Too high loading forces can lead to shear failure and too low loading forces can lead to (too) long duration of the experiment. Long duration of the loading like for the third experiment lead to results that are not expected to occur in practice, where the wave loads are not at the same point every time.
- For the sixth experiment 6 strain gauges were placed underneath the asphalt plate, because these can indicate the start of failure and can also help validate the results with back-calculations. Unfortunately 4 of these strain gauges got damaged due to corrosion during the experiment.

Experiments 1 and 2 were used in order to improve the test set up, so that the experiments conducted after these would give more accurate results. Therefore, in this report the experiment 1 and 2 are not considered for back-calculations.
Table 1 gives a summary of the conditions for each experiment.

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Relative density (%)</th>
<th>Loading force (kN)</th>
<th>Duration (s)</th>
<th># loadings</th>
<th>Loading head diameter (mm)</th>
<th>Temperature (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>11200</td>
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<td>20</td>
</tr>
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<td>19440</td>
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<td>3</td>
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<td>65.9</td>
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<td>343</td>
<td>100</td>
<td>10.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Step2: 7.1</td>
<td>3830</td>
<td>383</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Table 1: Overview of the conditions for each experiment [Deltares, 2012]*

The asphalt plate was made in the laboratory for tests 1 to 4 with identical properties of a new asphalt revetment. For the 5th and 6th test asphalt plates were produced out of a plate taken from an existing asphaltic dike revetment on a harbour dam at Groningen seaport which would be more likely to fail, as the stiffness and strength were found to be significantly lower than for the newly made asphalt. The asphalt concrete plates are around 40 years old and still of sufficient quality. The penetration of the bitumen of this asphalt is determined (20.5 (0.1 mm)) and has decreased in all these years (originally it is assumed to be between 70/100 and 80/100) [Deltares, 2012]. This can give a feeling for what the magnitude of the effect of aging might be. This means that the revetment may exert a stiffer behavior due to ageing, which will be checked by performing back-calculations (see next chapters).

*Figure 22: Cutting of a plate on the harbour dam at Groningen seaport [Deltares, 2012]*
The deformation of the asphalt surface was measured by using 6 displacement transducers (V1 to V6) mounted on top of the asphalt plate. The relative positions are illustrated in Figure 23 (note that the positions of V4 and V5 are different for experiment 6).

The diameter of the foot of the transducers is 6 mm. The transducers were mounted on a frame that was separated from that of the loading head. Transducer V3 was placed on a plate that is fixed to the loading head and measures its displacement. Transducer V5 was located close to the loading head position at a distance of 100 mm from the center so that the displacements could be measured at a close as possible distance to the center of the loading head. Transducers V2 and V4 are both at 200 mm from the center. Transducer V1 was placed on 350 mm distance from the center and transducer V6 was placed on the border of the asphalt plate at 27 mm from the asphalt plate edge and thus 418 mm from the center of the asphalt plate. It was not expected to lead to complications that the positions of the displacement transducers are either at the left or the right of the center, because the deformation of the asphalt plate was observed to be symmetrical at the end of the experiment. The displacement measurements for experiments have been recorded with a quick data logger and a slow data logger. The quick data logger measures 500 times per seconds and the slow data logger measures once in every 2 seconds. The accuracy of the displacement measurements is at least ± 0.01 mm for experiments 5 and 6 [Deltares, 2013]. For experiments 1 to 4 the accuracy is ± 0.05 mm [Deltares, 2012].
The measured displacements from both data loggers can be tracked on a computer screen during the experiments, which gives an idea of the gradual permanent deformation of the asphalt layer and the densification of the underlying sand bed as well as the response to the loading pulses. In case of sudden changes in these deformations the slow data logger failure of the asphalt plate might have occurred. The data from the quick data logger was used to create pulse diagrams for the wave attack period of 0.1 second. These plots of the force against displacement for a specific transducer (Lissajous plot) were used for further analysis of the experiments as to the amount of energy dissipation and effects of fatigue on this and also used during the experiment to help with making decisions regarding the progress of the test.
3.3 Predictions

Using finite element calculations and experiences from small scale laboratory tests carried out by Deltares, the proper dimensions of the test set-up and the testing conditions were defined for the medium scale tests. The wave attack is simplified to a series of pulse loadings with a duration of loading of 0.1 second and an interval between the subsequent pulses of a typical 10 second wave period.

The finite element calculations were also used to make a prediction of the deformations, stresses and strains and to interpret and explain the experimental results. During the medium scale tests, the deformations at the top surface of the asphalt were measured. During the experiment, no measurements were done under the asphalt or in the sand. Only after the loading was finished, the asphalt and the sand bed were investigated. It was expected that the gradual fatigue of the asphalt layer should be visible from the shape and magnitude of the deformations of the asphalt layer. That is why finite element calculations at several times during the experiment would illustrate more of what is happening between the asphalt concrete plate and the sand bed. Unfortunately it was not possible to model fatigue.

To make proper decisions about the magnitude of the applied loading pulses and the number of loadings, finite element calculations are useful. For experiment 5 on the old asphalt plate, these calculations were refined and adapted during the experiment. On the basis of these calculations the stresses and strains at the bottom of the asphalt can be estimated which indicate if the loading conditions will lead to failure within a certain number of repeated loadings.

The dimensions of the test set-up have also been checked with the help of FEM-calculations. A thickness of sand of 0.8 m would be enough to absorb the applied stresses according to these calculations [Deltares, 2012]. The rubber under the loading head has a Young’s modulus in the range of 9 to 11 MPa. The stiffness of the asphalt concrete depends on the temperature.
3.4 Inspection of asphalt slab

After each experiment, the asphalt plate was lifted and checked for cracks or other signs of failure (see Figure 24). Also the height of the sand surface is determined with a laser-scan and the density of the sand bed is determined by taking cores.

At the end of the experiments, the asphalt surface and the sand bed showed permanent deflections that were an order of magnitude larger than the deflections due to the loading pulses. These deflections were largest close to the loading head. This can be explained by the fact that in the area of sand right underneath the loading head most of the compaction of the sand occurs. During the pulse loadings the maximum elastic deflection of the asphalt was measured to be only parts of a millimeter. According to the estimated predictions failure would occur for experiments 3, 4, 5 and 6. The measured deformation was much larger than the original prediction, but did not lead to failure for the first 4 tests on the newly made asphalt. Experiments 5 and 6 were performed on old asphalt and showed signs of failure as cracks at the bottom of the asphalt were visible after lifting the asphalt plate (see Figure 25). During the fifth experiment itself, it was not clear from the measured displacements whether failure could have occurred. For experiment 5 the cracks had developed till the maximum depth of half of the asphalt plate thickness, while for experiment 6 only the beginning of the cracks was observed.
Chapter 4 Calculations with WESLEA

In this chapter the results will be shown including all relevant observations, tables and graphs. The approach with the WESLEA software will be treated first. Also the stiffness modulus and the fatigue curves will be determined to estimate the Miner sum for each of the experiments that are back-calculated.

4.1 Model design

The WESLEA software has originally been developed for the American Waterways Experiment Station (WES) of the US Army Corps of Engineers. WESLEA is a linear elastic multi-layer calculation software used for analysis of asphalt constructions consisting of a maximum of 5 layers. It was originally used for calculating the stresses and strains in an asphalt concrete layer for airport runways/landing strips. The program is also very useful for calculations of asphalt structures in road engineering. It is used to analyze the results of falling weight deflection measurements. The method and schematization can be compared with the medium scale tests.

Since the WESLEA program has been developed for the American system of units, the input in SI-units will be converted to American units for processing. This will cause some rounding errors for the output results. The effect of these errors are not taken into account, since the use of this program is only to get a feeling for what is happening during the experiments and to produce only rough (first) results.

The coordinate system used in the WESLEA software is given in Figure 26. Tensile stresses have negative values and compressive stresses positive values in the output of the program.

![Figure 26: Coordinate system used in WESLEA software](image)
With the WESLEA program the maximum elastic deformation for a selection of pulse loadings measured during the experiments are fitted. For the back-calculations only the measurements from transducers V3, V5, V4, V2 and V1 are taken into account (see Figure 27), because the WESLEA software assumes an infinitely long asphalt concrete slab for calculations (in the x- and y-direction). Transducer V6 is not taken into account for these calculations, because it is too close to the edge of the asphalt plate.

![Figure 27: Position of displacement transducers (V1 to V6) on the asphalt slab](image)

A correction needs to be taken into account regarding the rubber underneath the loading head. Since transducer V3 is mounted on the loading head, the measurements of this transducer will include the elastic deformation of the rubber. The correction due to the rubber is determined as given in APPENDIX D and is already applied to the measured values of the elastic deformations in the tables in section 4.3.
The geometry of the model in WESLEA is built out of 5 layers (see Figure 28):

1. 50 mm for the asphalt layer on top.
2. 100 mm for the first sand layer.
3. 200 mm for the second sand layer.
4. 500 mm for the third sand layer.
5. An infinitely thick layer. This is a standard assumption which could not be changed in this program as the use is originally for modeling in situ cases. The largest possible stiffness (68947.6 MPa) is used to resemble the bottom of the cylinder on the concrete floor in the lab.

The sand is divided into 3 separate layers in order to cope with nonlinear behavior like compaction of the upper layer(-s) of sand and stress dependent elastic behavior of the sand. The thickness of the upper layers is smaller, because the compaction of the sand is assumed to happen over a layer of 100 to 200 mm.

The loading force is adjusted for each experiment and loading. The contact area of the loading head (with a diameter of 100 mm) is given through input for tire pressure. The tire pressure is used in this software, because it was originally designed for roads and runways. The tire pressure input is calculated by dividing the load by the contact area of the loading head.
4.2 Fitting the measured elastic deflections

The elastic deformations measured from the medium scale experiments conducted by Deltares have been fitted with the help of WESLEA software for the selected measurement points of experiment 3, 4, 5 and 6 (see APPENDIX B for the measurements). The calculated elastic deflections have to match the measured elastic deflections close to the center of the asphalt plate. That means that the input values, which are the layer stiffness modulus, will be found iteratively. Therefore criteria for the fit have to be set and the accuracy of the measurements needs to be known.

The fitting is done iteratively by giving input values for the elastic modulus (in MPa) of each layer. The default Poisson’s ratio of 0.35 is used for modeling the asphalt concrete and 0.25 for the Baskarp sand as agreed on by Deltares and KOAC-NPC. In total 8 points are analyzed with the WESLEA model, namely the top and the bottom of the asphalt layer at the measuring points V3, V5, V4, V2 and V1. During the experiments the top of the asphalt layer is monitored, so the top of the asphalt is used to get a fit. The maximum strains are expected to be along the bottom of the asphalt layer, which makes the bottom also important to take into consideration for analysis of the fatigue behavior.

Important criteria that should be possible to meet for the accuracy while fitting the elastic deflections to the results of the experiments are:

- The difference between measured and calculated deflections of individual points should be less than 10% with respect to the measured value or less than the uncertainty of displacement measurements. For the experiments 1 to 4 the uncertainty of the measurements was 50 μm [Deltares, 2012]. During experiments 5 and 6 a better amplifier was used which reduced the uncertainty of the measurements to 10 μm [Deltares, 2013]. This means that in case the measurement is smaller than 500 μm, the uncertainty of the measurement is higher than the 10% difference for experiments 1 to 4. For experiments 5 and 6 this is only the case if the measurements are smaller than 100 μm.
- The Surface Curvature Index (SCI) of two measured/calculated points (V5 and V2V4) with respect to the measurement of transducer V3 must be less than 10%. This criteria is to be met, because in some cases the points might be within the boundary of the measurement uncertainty or 10% difference, but the curvature of the profile might be too steep or gentle. This would lead to too high or low strains for the fit.

A procedure which can be followed for making a fit is described in the APPENDIX C. In general the highest possible asphalt stiffness modulus in combination with the lowest possible sand stiffness modulus are sought for. In the following paragraphs the results of the fitting are compared to the measured elastic displacements. The maximum strains along the bottom and top of the asphalt layer are also calculated, which can be used to estimate the maximum number of loadings until failure for the asphalt concrete plate on sand.
4.2.1 Experiment 3

A total of 3 fits are made for experiment 3. The load of 12.2 kN was kept constant during the whole test, that is the reason why 3 points in time were chosen. These are at 10 minutes, 10 hours and at the end of the test at 51 hours after start of the experiment. These 3 points in time are chosen because these represent the 3 stages where the measured deflections indicate significant changes in the profile as the experiment goes on (see Figure 29).

![Figure 29: Vertical elastic displacements-experiment 3- total [Deltares, 2012]](image)

The maximum elastic deformations measured for the first 60 minutes almost coincide as seen in Figure 29. This can be marked as a first stage. From the measured elastic displacements illustrated in the figure above it can be roughly observed that in time the measurements from transducers V3, V2, V4, V1 and V6 tend to become smaller while V5 becomes larger. Note that in figure 29 the corrections for the rubber have not been applied for the measurements with transducer V3.
4.2.1.1 First stage at \( t = 10 \) minutes

At \( t = 10 \) minutes in the first stage of loading the sand underneath the asphalt concrete plate is expected to show some signs of compaction (higher stiffness modulus). The table below gives an overview of the measured maximum displacements after 10 minutes after the start of experiment 3.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>maximum elastic displacement (( \mu m ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 2: Measured maximum elastic deformation at \( t = 10 \) minutes for experiment 3*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 1440 MPa  
2. First sand layer: 82 MPa  
3. Second sand layer: 42 MPa  
4. Third sand layer: 40 MPa  
5. Bottom layer: 68947.6 MPa

Compaction of the upper sand layer can be identified with the higher stiffness found for the input layer of the first sand layer. The asphalt concrete has a relatively lower stiffness than expected (see section 4.4.1.2). It could be that the new asphalt concrete expresses lower stiffness as the sand takes on a larger part of the loading. Table 3 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (( \mu m ))</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>134.31</td>
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<td>73.8</td>
<td>-23.62</td>
</tr>
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</table>

*Table 3: Results from calculations for experiment 3 at \( t = 10 \) minutes*

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest strains. The maximum strains are indicated in bold in Table 3.
In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 30.

**Figure 30: Measured elastic deformations vs. calculated elastic deformations from fitting at t= 10 minutes of experiment 3**

From Figure 30 it is observed that a good fit is achieved given that a higher stiffness is used for the first sand layer (possibly due to compaction of this layer).
4.2.1.2 Second stage at t= 10 hours

At t= 10 hours the second stage of loading is marked where the sand underneath the asphalt concrete plate is expected to show more signs of compaction. The increasing permanent deformations can also affect the maximum elastic deformation of the asphalt concrete plate. The table below gives an overview of the measured maximum displacements after 10 hours after the start of experiment 3.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

_Table 4: Measured maximum elastic deformation at t= 10 hours for experiment 3_

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 2700 MPa
2. First sand layer: 30 MPa
3. Second sand layer: 60 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68947.6 MPa

Compared to t= 10 minutes, at t= 10 hours the asphalt concrete has a relatively higher stiffness while the first sand layer has a very low stiffness. This can be explained by the formation of a hole (or space) between the asphalt concrete layer and the sand due to compaction of sand and the increasing permanent deformations. The asphalt concrete responds with a higher stiffness to the loading as this develops. Table 5 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
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<td>208.82</td>
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<td>-198.82</td>
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<tr>
<td>350</td>
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<td>124.17</td>
<td>56.62</td>
<td>-110.77</td>
<td>29.16</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>124.25</td>
<td>-48.46</td>
<td>111.24</td>
<td>-32.59</td>
</tr>
</tbody>
</table>

_Table 5: Results from calculations for experiment 3 at t= 10 hours_
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest strains. The maximum strains are indicated in bold in Table 5. It is noticed that these strains are smaller than those found for the fit at $t=10$ minutes. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 31.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

**Figure 31: Measured elastic deformations vs. calculated elastic deformations from fitting at $t=10$ hours of experiment 3**

From Figure 31 it is observed that a reasonable fit is achieved choosing the highest possible stiffness modulus for the asphalt concrete plate. Compaction of the sand and possibly plastic deformations of the asphalt concrete can be detected. This nonlinear behavior is largest in the area within a radius between 100 to 200 mm from the center. Further from this area it is expected that the asphalt and the sand have a more linear elastic behavior.
4.2.1.3 At t= 51 hours (close to the end of experiment 3)

At t= 51 hours the final stage of loading is marked where the most compaction of the sand and nonlinear deformation of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum displacements after 51 hours after the start of experiment 3.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 6: Measured maximum elastic deformation at t= 51 hours for experiment 3*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 5300 MPa
2. First sand layer: 21 MPa
3. Second sand layer: 40 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68947.6 MPa

Compared to t= 10 minutes and t= 10 hours, at t= 51 hours the asphalt concrete has a relatively higher stiffness while the first and the second sand layer have a very low stiffness modulus. This can be explained by the formation of a hole (or space) between the asphalt concrete layer and the sand due to compaction and the permanent deformation. The asphalt concrete responds with a higher stiffness after more loadings. Table 7 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>630.15</td>
<td>597.47</td>
<td>597.47</td>
<td>-460.86</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>623.59</td>
<td><strong>-649.43</strong></td>
<td><strong>-649.43</strong></td>
<td><strong>709.61</strong></td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>509.92</td>
<td>364.22</td>
<td>94.51</td>
<td>-247</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>509.97</td>
<td>-373.27</td>
<td>-72.6</td>
<td>246.6</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>349.75</td>
<td>177.48</td>
<td>-70.79</td>
<td>-57.44</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>349.78</td>
<td>-175.53</td>
<td>77.93</td>
<td>56.1</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>171.42</td>
<td>63.23</td>
<td>-87.95</td>
<td>13.31</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>171.44</td>
<td>-60.59</td>
<td>89.14</td>
<td>-14.2</td>
</tr>
</tbody>
</table>

*Table 7: Results from calculations for experiment 3 at t= 51 hours*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest strains. The maximum strains are indicated in bold in Table 7. These strains are smaller than those found for the fits after t= 10 minutes and t= 10 hours. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 32.

**Figure 32: Measured elastic deformations vs. calculated elastic deformations from fitting at t= 51 hours of experiment 3**

From Figure 32 it is observed that a poor fit is achieved, but still the highest possible asphalt modulus is used here. The shape of the profile, especially for the measurement with transducer V3, is peculiar. The correction of the rubber is possibly too large, because in the most extreme case a flat part in the profile for which the measurements of transducers V3 and V5 are equal is expected. A possible explanation for this will be treated later on in this report. The effects of nonlinear aspects like compaction of the sand and nonlinear viscoelastic behavior of the asphalt concrete have increased. Nonlinear behavior can be observed as the permanent displacements increase. The nonlinear behavior is largest in the area within a radius between 100 to 200 mm from the center. Further from this area it is expected that the asphalt and the sand have a more linear elastic behavior. This area tends to become larger, the longer the loading time is.
4.2.2 Experiment 4
A total of 6 fits are made for experiment 4. The fitting of the elastic deformations cannot be achieved for the loadings of 16.25 kN and 19.85 kN, because for the WESLEA software the pressures are too high. This is because the program assumes that the loads are transferred by tires, which have certain limitations. For the other 3 loads namely 2.25 kN, 4.3 kN and 12.4 kN the pressure is small enough and data is known for the first loadings and the last loadings. This makes a total of 6 fits that can be performed. Each stage of loading is made clear with Figure 33.

Figure 33: Vertical elastic displacements-experiment 4 [Deltares, 2012]

The maximum elastic deformations measured increase as the loads increase, but for 2.25 kN, 4.3 kN and 19.85 kN they decrease in time for the same load magnitudes. This is possibly caused by compaction of the sand underneath the asphalt concrete slab. Note that in figure 33 the corrections for the rubber have not been applied yet for the measurements with transducer V3.
4.2.2.1 \( p_{\text{first}} = 2.25 \text{ kN} \)

For the first pulse of 2.25 kN the sand underneath the asphalt concrete may be compacted during the test pulses, but no nonlinear (visco-elastic) behavior of the asphalt concrete plate is expected to be observed. The table below gives an overview of the measured maximum displacements for the first loadings of 2.25 kN of experiment 4.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 8: Measured maximum elastic deformation for \( p_{\text{first}} = 2.25 \text{ kN} \) for experiment 4*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 1550 MPa
2. First sand layer: 60 MPa
3. Second sand layer: 60 MPa
4. Third sand layer: 48 MPa
5. Bottom layer: 68947.6 MPa

The asphalt concrete has a relatively lower stiffness than expected (see section 4.4.1.2). Also here it could be that the new asphalt expresses lower stiffness as the sand takes on a larger part of the loading. The first and second sand layers have relatively high stiffness moduli, possibly caused by compaction of these layers during the test phase. Table 9 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>128.46</td>
<td>223.2</td>
<td>223.2</td>
<td>-125.23</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>124.22</td>
<td><strong>-238.82</strong></td>
<td><strong>-238.82</strong></td>
<td>279.54</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>83.79</td>
<td>107.92</td>
<td>-13.2</td>
<td>-51</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>83.93</td>
<td>-102.71</td>
<td>30.98</td>
<td>46.38</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>45.82</td>
<td>36.55</td>
<td>-38.47</td>
<td>1.04</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>45.92</td>
<td>-29.39</td>
<td>41.84</td>
<td>-4.92</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>19.56</td>
<td>9.25</td>
<td>-16.55</td>
<td>3.93</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>19.59</td>
<td>-5.05</td>
<td>15.11</td>
<td>-5.32</td>
</tr>
</tbody>
</table>

*Table 9: Results from calculations for \( p_{\text{first}} = 2.25 \text{ kN} \) for experiment 4*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. The maximum strains are indicated in bold in Table 9. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 34.

**Figure 34: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{\text{first}} = 2.25$ kN for experiment 4**

From Figure 34 it is observed that a reasonable fit is achieved, given that a higher stiffness is used for the first and second sand layer (possibly due to compaction of these layers also during the test pulses).
4.2.2.2 \( p_{\text{last}} = 2.25 \text{ kN} \)

For the last loadings of 2.25 kN more compaction of the sand underneath the asphalt concrete plate is expected to be observed. The table below gives an overview of the measured maximum displacements for the last loadings of 2.25 kN of experiment 4.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>105</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>53</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>41</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>23</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
<tr>
<td></td>
<td>7</td>
</tr>
</tbody>
</table>

*Table 10: Measured maximum elastic deformation for \( p_{\text{last}} = 2.25 \text{ kN for experiment 4})*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 3300 MPa
2. First sand layer: 54 MPa
3. Second sand layer: 52 MPa
4. Third sand layer: 52 MPa
5. Bottom layer: 68947.6 MPa

Compared to the first measurement the asphalt concrete stiffness input is approximately 2 times larger while the stiffness of the (first and second) sand layers decreased. This decrease of the stiffness modulus of the upper sand layers is not expected. It is possible that the asphalt concrete plate takes on more of the imposed stresses in time, which results in a higher asphalt stiffness and lower sand stiffness. Table 11 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>104.3</td>
<td>136.61</td>
<td>136.61</td>
<td>-93.04</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>102.36</td>
<td>-147.42</td>
<td>-147.42</td>
<td>165.21</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>77</td>
<td>74.49</td>
<td>5.74</td>
<td>-43.2</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>77.05</td>
<td>-73.88</td>
<td>2.49</td>
<td>41.39</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>47.51</td>
<td>30.39</td>
<td>-22.2</td>
<td>-4.4</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>47.55</td>
<td>-27.76</td>
<td>24.38</td>
<td>2.87</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>21.42</td>
<td>8.88</td>
<td>-15.2</td>
<td>3.41</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>21.43</td>
<td>-7.11</td>
<td>14.86</td>
<td>-4</td>
</tr>
</tbody>
</table>

*Table 11: Results from calculations for \( p_{\text{last}} = 2.25 \text{ kN for experiment 4})*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. The maximum strains have become smaller than the first loading (indicated in bold in Table 11). In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 35.

![Figure 35: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{last} = 2.25$ kN for experiment 4](image)

From Figure 35 it is observed that the fit is reasonable and still within boundaries. The measurements with transducer V5 deviate most from the calculated elastic deformation profile. There is a possibility that the measurements with transducer V5 deviate from occurring deflections. Therefore a fit is made excluding this measurement. After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 3700 MPa
2. First sand layer: 49 MPa
3. Second sand layer: 49 MPa
4. Third sand layer: 49 MPa
5. Bottom layer: 68947.6 MPa

Compared to the previous fit the asphalt concrete stiffness input is slightly larger while the stiffness moduli of the sand layers slightly decreased. Table 12 gives an overview of the results from calculations with the above mentioned fit.
<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>104.99</td>
<td>128.43</td>
<td>128.43</td>
<td>-90.07</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>103.26</td>
<td>-138.74</td>
<td>-138.74</td>
<td>154.5</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>79.3</td>
<td>71.71</td>
<td>8.33</td>
<td>-43.09</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>79.34</td>
<td>-71.57</td>
<td>-1.12</td>
<td>41.61</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>50.28</td>
<td>30.4</td>
<td>-20.01</td>
<td>-5.58</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>50.31</td>
<td>-28.26</td>
<td>22.06</td>
<td>4.28</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>23.26</td>
<td>9.27</td>
<td>-15.23</td>
<td>3.21</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>23.28</td>
<td>-7.75</td>
<td>15.05</td>
<td>-3.75</td>
</tr>
</tbody>
</table>

Table 12: Results from calculations for $p_{last}=2.25$ kN for experiment 4 without V5

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. The maximum strains are slightly smaller than the previous fit (indicated in bold in Table 12). In order to compare the measured maximum elastic deflections to the calculated fit the values are plotted and illustrated in Figure 36.

![Figure 36: Measured elastic deformations without V5 vs. calculated elastic deformations from fitting $p_{last}=2.25$ kN for experiment 4](image)

From Figure 36 it is observed that a better fit can be obtained if the measurement with V5 is excluded. Since it is the second closest transducer to the center, it is very unfortunate not to take it into account for back-calculation. Excluding the measurements with transducer V5 obviously makes the fitting process less complicated and possibly less accurate. However the stiffness moduli and strains do not differ very much.
4.2.2.3 \( p_{\text{first}} = 4.3 \, \text{kN} \)

For the first loadings of 4.3 kN not much more compaction of the sand underneath the asphalt concrete plate is expected to be observed. The table below gives an overview of the measured maximum displacements for the first loadings of 4.3 kN of experiment 4.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (( \mu \text{m} ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 13: Measured maximum elastic deformation for \( p_{\text{first}} = 4.3 \, \text{kN} \) for experiment 4*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 2800 MPa
2. First sand layer: 50 MPa
3. Second sand layer: 50 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68947.6 MPa

Compared to the last loadings of 2.25 kN, the stiffness of the sand and asphalt concrete have decreased. This was not expected, because an increase of the load would normally cause a higher stiffness modulus of the sand layers. Table 14 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>( y ): distance from center (mm)</th>
<th>( z ): vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (( \mu \text{m} ))</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>218.8</td>
<td>299.21</td>
<td>299.21</td>
<td>-200.41</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>214.41</td>
<td>-323.11</td>
<td>-323.11</td>
<td>363.28</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>158.99</td>
<td>160.91</td>
<td>8.59</td>
<td>-91.26</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>159.1</td>
<td>-159.34</td>
<td>9.85</td>
<td>87.35</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>96.08</td>
<td>64.05</td>
<td>-49.95</td>
<td>-7.59</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>96.17</td>
<td>-58.13</td>
<td>54.64</td>
<td>4.2</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>42.25</td>
<td>18.1</td>
<td>-32.12</td>
<td>7.55</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>42.28</td>
<td>-14.18</td>
<td>31.26</td>
<td>-8.86</td>
</tr>
</tbody>
</table>

*Table 14: Results from calculations for \( p_{\text{first}} = 4.3 \, \text{kN} \) for experiment 4*

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 14). The maximum strains are larger than for the last loadings of 2.25 kN, which is expected if the load increases.
In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 37.

**Figure 37: Measured elastic deformations vs. calculated elastic deformations from fitting \( p_{\text{fit}} = 4.3 \) kN for experiment 4**

From Figure 37 it is observed that a poor fit is achieved, but still within boundaries. Here there also is a possibility that the measurements with transducer V5 deviate from occurring deflections, because the magnitude of the measurements with transducer V5 are close to those with transducers V2 and V4. Therefore a fit is made excluding this measurement. After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 3700 MPa
2. First sand layer: 40 MPa
3. Second sand layer: 40 MPa
4. Third sand layer: 40 MPa
5. Bottom layer: 68947.6 MPa

Compared to the previous fit the asphalt concrete stiffness input is (about 30%) larger while the stiffness of the sand layers decreased. Compared to the last loadings of 2.25 kN, the stiffness of the asphalt concrete has the same magnitude while the stiffness modulus of the sand layers have decreased. A decrease of the stiffness modulus of the sand layers is not expected for an increased load. Table 15 gives an overview of the results from calculations with the above mentioned fit.
<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>228.28</td>
<td>258.38</td>
<td>258.38</td>
<td>-186.07</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>225</td>
<td><strong>-279.11</strong></td>
<td><strong>-279.11</strong></td>
<td>309.18</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>176.58</td>
<td>147.73</td>
<td>23.05</td>
<td>-91.95</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>176.66</td>
<td>-148.1</td>
<td>-9.43</td>
<td>89.22</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>115.48</td>
<td>65.22</td>
<td>-37.76</td>
<td>-14.78</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>115.54</td>
<td>-61.43</td>
<td>41.84</td>
<td>12.35</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>55.44</td>
<td>20.96</td>
<td>-32.32</td>
<td>6.12</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>55.46</td>
<td>-18.14</td>
<td>32.18</td>
<td>-7.16</td>
</tr>
</tbody>
</table>

Table 15: Results from calculations for $p_{\text{first}} = 4.3$ kN for experiment 4 without V5

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 15). The maximum strains have the same magnitude but slightly smaller than the previous fit. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 38.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

**Figure 38:** Measured elastic deformations without V5 vs. calculated elastic deformations from fitting $p_{\text{first}} = 4.3$ kN for experiment 4

From Figure 38 it is observed that a better fit can be obtained if the measurement with V5 is excluded. Since it is the second closest transducer to the center, it is very unfortunate not to take it into account for back-calculation. Excluding the measurements with transducer V5 obviously makes the fitting process less complicated and possibly less accurate. The strains do not differ very much, which is not the case for the stiffness moduli.
4.2.2.4 $p_{\text{last}}= 4.3 \text{ kN}$

For the last loadings of 4.3 kN more compaction of the sand underneath the asphalt concrete plate and/or larger deviations measured by transducer V5 are expected to be observed. Also some nonlinear (visco-elastic) behavior of the asphalt concrete can occur here. The table below gives an overview of the measured maximum displacements for the last loadings of 4.3 kN of experiment 4.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (µm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>220</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>101</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>98</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>43</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
<tr>
<td></td>
<td>12</td>
</tr>
</tbody>
</table>

*Table 16: Measured maximum elastic deformation for $p_{\text{last}}= 4.3 \text{ kN}$ for experiment 4*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 4000 MPa
2. First sand layer: 50 MPa
3. Second sand layer: 50 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68947.6 MPa

The asphalt concrete has a relatively higher stiffness than for the first loads of 4.3 kN, which is also observed for the first and last load of 2.25 kN. The sand has the same stiffness values as for the fitting of the first loadings of 4.3 kN and the loadings of 2.25 kN with V5. Table 17 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (µm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>192.56</td>
<td>230.44</td>
<td>230.44</td>
<td>-162.9</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>189.51</td>
<td>-248.95</td>
<td>-248.95</td>
<td>276.79</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>146.47</td>
<td>129.55</td>
<td>16.55</td>
<td>-78.67</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>146.54</td>
<td>-129.48</td>
<td>-3.84</td>
<td>76.07</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>93.7</td>
<td>55.57</td>
<td>-35.32</td>
<td>-10.9</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>93.76</td>
<td>-51.87</td>
<td>38.98</td>
<td>8.61</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>43.8</td>
<td>17.2</td>
<td>-27.77</td>
<td>5.7</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>43.83</td>
<td>-14.54</td>
<td>27.51</td>
<td>-6.65</td>
</tr>
</tbody>
</table>

*Table 17: Results from calculations for $p_{\text{last}}= 4.3 \text{ kN}$ for experiment 4*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 17). The maximum strains are a bit lower than for the first loadings of 4.3 kN. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 39.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

*Figure 39: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{\text{last}} = 4.3$ kN for experiment 4*

From the figure above it is observed that the fit is not perfect, but still within boundaries of the criteria for individual points. The SCI is larger than 10%. Here there also is a possibility that the measurements with transducer V5 deviate from occurring deflections, especially as the measurements of V2, V4 and V5 are of the same magnitude. A fit is made excluding the measurement of V5. After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 2900 MPa
2. First sand layer: 49 MPa
3. Second sand layer: 49 MPa
4. Third sand layer: 49 MPa
5. Bottom layer: 68947.6 MPa

Compared to the previous fit the asphalt concrete stiffness input is (about 30%) smaller while the stiffness of the sand layers remains of the same magnitude. Table 18 gives an overview of the results from calculations with the above mentioned fit.
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 18). The maximum strains are slightly higher than for the previous fit. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 40.

**Figure 40: Measured elastic deformations without V5 vs. calculated elastic deformations from fitting $p_{\text{last}} = 4.3$ kN for experiment 4**

From the figure above it is observed that a better fit can be obtained if the measurement with V5 is excluded. Since it is the second closest transducer to the center, it is very unfortunate not to take it into account for back-calculation. Excluding the measurements with transducer V5 obviously makes the fitting process less complicated and possibly less accurate.
4.2.2.5 \( p_{\text{first}} = 12.4 \) kN

For the first loadings of 12.4 kN more compaction of the sand underneath the asphalt concrete plate and/or larger deviations measured by transducer V5 as seen for the 4.3 kN loadings are expected to be observed. Also some nonlinear (visco-elastic) behavior of the asphalt concrete plate is expected. The table below gives an overview of the measured maximum displacements for the first loadings of 12.4 kN of experiment 4.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (( \mu \text{m} ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 19: Measured maximum elastic deformation for \( p_{\text{first}} = 12.4 \) kN for experiment 4*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 3600 MPa
2. First sand layer: 50 MPa
3. Second sand layer: 78 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68947.6 MPa

Compaction of the second sand layer can be identified with the higher stiffness found for the input layer of the second sand layer. The first sand layer has a lower stiffness value possibly because some space formed right underneath the asphalt plate below the loading head for which a combined stiffness modulus is found caused by compaction and the increasing permanent deformation. Table 20 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>( y ): distance from center (mm)</th>
<th>( z ): vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (( \mu \text{m} ))</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>526.26</td>
<td>696.47</td>
<td>696.47</td>
<td>-476.84</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>516.41</td>
<td>-755.7</td>
<td>-755.7</td>
<td>844.68</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>386.91</td>
<td>379</td>
<td>26.73</td>
<td>-218.46</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>387.12</td>
<td>-379.05</td>
<td>13.57</td>
<td>211.79</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>236.95</td>
<td>152.68</td>
<td>-116.95</td>
<td>-19.23</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>237.13</td>
<td>-140.97</td>
<td>129.07</td>
<td>11.89</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>107.18</td>
<td>43.39</td>
<td>-76.27</td>
<td>17.71</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>107.29</td>
<td>-34.61</td>
<td>76.45</td>
<td>-21.74</td>
</tr>
</tbody>
</table>

*Table 20: Results from calculations for \( p_{\text{first}} = 12.4 \) kN for experiment 4*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 20). The maximum strains are higher than for the loadings of 2.25 and 4.3 kN, which was not unforeseeable. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 41.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

**Figure 41: Measured elastic deformations vs. calculated elastic deformations from fitting \( p_{\text{first}} = 12.4 \) kN for experiment 4**

From Figure 41 it is observed that the fit is not perfect, but still within boundaries of the criteria for individual points. The SCI is larger than 10%. Here there also is a possibility that the measurements with transducer V5 deviate from occurring deflections, especially as the measurements of V2, V4 and V5 are of the same magnitude. A fit is made excluding the measurement of V5. After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 3800 MPa
2. First sand layer: 48 MPa
3. Second sand layer: 48 MPa
4. Third sand layer: 48 MPa
5. Bottom layer: 68947.6 MPa

The stiffness modulus of the asphalt concrete is of the same magnitude as the previous fit, while the stiffness modulus of the sand is slightly lower and equal for all layers. Table 21 gives an overview of the results from calculations with the above mentioned fit.
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 21). The maximum strains are of the same magnitude of the previous fit. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 42.

From Figure 42 it is observed that a better fit can be obtained if the measurement with V5 is excluded. Since it is the second closest transducer to the center, it is very unfortunate not to take it into account for back-calculation. Excluding the measurements with transducer V5 obviously makes the fitting process less complicated and possibly less accurate.
For the last loadings of 12.4 kN more compaction of the sand underneath the asphalt concrete plate, more nonlinear (visco-elastic) behavior of the asphalt concrete plate and/or larger deviations measured by transducer V5 as seen for the previous loadings are expected to be observed. The table below gives an overview of the measured maximum displacements for the last loadings of 12.4 kN of experiment 4.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 22: Measured maximum elastic deformation for $p_{\text{last}} = 12.4$ kN for experiment 4*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 7000 MPa
2. First sand layer: 50 MPa
3. Second sand layer: 50 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68947.6 MPa

The stiffness modulus of the asphalt concrete is larger than for the earlier loadings. Again it can be observed that it is possible that the asphalt concrete plate takes on more stress in time, which results in a higher asphalt stiffness. The second sand layer has a lower stiffness modulus of 50 MPa again. However, this is of the same magnitude of the fit without V5. Table 23 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>451.45</td>
<td>434.08</td>
<td>434.08</td>
<td>-326.96</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>446.47</td>
<td>-468.47</td>
<td>-468.47</td>
<td>514.7</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>364.45</td>
<td>259.47</td>
<td>59.75</td>
<td>-171.88</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>364.57</td>
<td>-261.88</td>
<td>-39.71</td>
<td>168.09</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>252.79</td>
<td>123.65</td>
<td>-52.73</td>
<td>-38.18</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>252.87</td>
<td>-118.84</td>
<td>59.33</td>
<td>34.72</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>131.22</td>
<td>44.2</td>
<td>-58.41</td>
<td>7.65</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>131.26</td>
<td>-40.2</td>
<td>58.83</td>
<td>-9.28</td>
</tr>
</tbody>
</table>

*Table 23: Results from calculations for $p_{\text{last}} = 12.4$ kN for experiment 4*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 23). The maximum strains are lower than for the first loadings of 12.4 kN. The decrease of the strains together with the increase of asphalt concrete stiffness is also observed for the 2.25 kN loading. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 43.

**Vertical elastic deformations: measured vs. calculated through fitting**

![Graph showing measured vs. calculated elastic deformations](image)

*Figure 43: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{last} = 12.4$ kN for experiment 4*

From Figure 43 it is observed that the fit is good, with higher elastic displacements fitted for V5. Here there also is a possibility that the measurements with transducer V5 deviate from occurring deflections. A fit is made excluding this measurement. After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 6800 MPa
2. First sand layer: 49 MPa
3. Second sand layer: 49 MPa
4. Third sand layer: 49 MPa
5. Bottom layer: 68947.6 MPa

The stiffness moduli of the asphalt concrete and the sand layers are of the same magnitude of the previous fit. Table 24 gives an overview of the results from calculations with the above mentioned fit.
<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (µm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>462.19</td>
<td>445.97</td>
<td>445.97</td>
<td>-335.64</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>457.06</td>
<td><strong>-481.32</strong></td>
<td><strong>-481.32</strong></td>
<td><strong>528.89</strong></td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>372.81</td>
<td>266.34</td>
<td>60.94</td>
<td>-176.22</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>372.93</td>
<td>-268.79</td>
<td>-40.29</td>
<td>172.31</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>258.28</td>
<td>126.73</td>
<td>-54.43</td>
<td>-38.92</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>258.37</td>
<td>-121.75</td>
<td>61.22</td>
<td>35.35</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>133.85</td>
<td>45.21</td>
<td>-59.95</td>
<td>7.94</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>133.89</td>
<td>-41.08</td>
<td>60.37</td>
<td>-9.62</td>
</tr>
</tbody>
</table>

*Table 24: Results from calculations for $p_{last} = 12.4$ kN for experiment 4 without V5*

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains (indicated in bold in Table 24). The maximum strains are lower than for the first loadings of 12.4 kN and slightly larger than the previous fit. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 44.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

*Figure 44: Measured elastic deformations without V5 vs. calculated elastic deformations from fitting $p_{last} = 12.4$ kN for experiment 4*

From Figure 44 it is observed that a better fit can be obtained if the measurement with V5 is excluded.
4.2.3 Experiment 5

A total of 5 fits are made for experiment 5, because with the WESLEA software the profile of the elastic deformation could not be fitted properly for the first loadings of 2.1 kN. The 5 loading stages that are fitted are: the last loadings of 2.4 kN, the first loadings of 5.7 kN, the last loadings of 5.9 kN, the first loadings of 8.5 kN and the last loadings of 8.7 kN. Each stage of loading is made clear with Figure 45.

![Experiment old asphalt - day 2](image)

*Figure 45: Vertical elastic displacements-experiment 5 [Deltares, 2013]*

It is observed that the maximum elastic displacements measured increase as the loads increase, but decrease in time for the same load magnitudes.
### 4.2.3.1 \( p_{last} = 2.4 \, kN \)

For the last loadings of 2.4 kN not much compaction of the sand underneath the asphalt concrete plate and nonlinear (visco-elastic) behavior of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum displacements for the last loadings of 2.4 kN of experiment 5.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>200</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>55</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>20</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>

*Table 25: Measured maximum elastic deformation for \( p_{last} = 2.4 \, kN \) for experiment 5*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 580 MPa
2. First sand layer: 53 MPa
3. Second sand layer: 53 MPa
4. Third sand layer: 50 MPa
5. Bottom layer: 68,947.6 MPa

The asphalt concrete has a very low stiffness. All the sand layers have the same magnitude for the values of the layer stiffness. Table 26 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>200.58</td>
<td>453.33</td>
<td>453.33</td>
<td>-158.9</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>187.75</td>
<td>-474.6</td>
<td>-474.6</td>
<td>609.49</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>108.09</td>
<td>188.95</td>
<td>-68.27</td>
<td>-64.97</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>108.47</td>
<td>-167.68</td>
<td>111.1</td>
<td>54.21</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>50.6</td>
<td>52.6</td>
<td>-69.93</td>
<td>9.34</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>50.85</td>
<td>-32.95</td>
<td>72.85</td>
<td>-18.43</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>20.09</td>
<td>12.49</td>
<td>-20.91</td>
<td>4.54</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>20.16</td>
<td>-2.58</td>
<td>15.31</td>
<td>-6.86</td>
</tr>
</tbody>
</table>

*Table 26: Results from calculations for \( p_{last} = 2.4 \, kN \) for experiment 5*

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. The maximum strains are indicated in bold in Table 26. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 46.
Figure 46: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{\text{inelast}} = 2.4$ kN for experiment 5

From Figure 46 it is observed that a good fit is achieved. However the very low stiffness for the asphalt concrete can point to some nonlinear behavior of the asphalt concrete plate.
### 4.2.3.2 $p_{\text{first}} = 5.7$ kN

For the first loadings of 5.7 kN not much compaction of the sand underneath the asphalt concrete plate and nonlinear (visco-elastic) behavior of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum displacements for the first loadings of 5.7 kN of experiment 5.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>550</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>100</td>
</tr>
<tr>
<td>V1</td>
<td>200</td>
</tr>
<tr>
<td>V6</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 27: Measured maximum elastic deformation for $p_{\text{first}} = 5.7$ kN for experiment 5*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 800 MPa
2. First sand layer: 37 MPa
3. Second sand layer: 36 MPa
4. Third sand layer: 36 MPa
5. Bottom layer: 68947.6 MPa

The asphalt concrete has a very low stiffness, slightly higher than for the last load of 2.4 kN. All the sand layers have a lower stiffness value than for the loadings of 2.4 kN. This was not expected since the load increased. Table 28 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>548</td>
<td>1026.44</td>
<td>1026.44</td>
<td>-540.25</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>526.9</td>
<td>-1097.69</td>
<td>-1097.69</td>
<td>1302.66</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>341.9</td>
<td>481.47</td>
<td>-83.7</td>
<td>-214.17</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>342.51</td>
<td>-455.22</td>
<td>167.03</td>
<td>194.33</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>177.04</td>
<td>155.08</td>
<td>-177.84</td>
<td>12.26</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>177.47</td>
<td>-121.82</td>
<td>190.13</td>
<td>-28.69</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>70.44</td>
<td>37</td>
<td>-70.31</td>
<td>17.94</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>70.57</td>
<td>-18.69</td>
<td>61.68</td>
<td>-22.84</td>
</tr>
</tbody>
</table>

*Table 28: Results from calculations for $p_{\text{first}} = 5.7$ kN for experiment 5*

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. As expected the maximum strains are much higher than for the last loadings of 2.4 kN (indicated in bold in Table 28).
In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 47.

\[ p_{\text{ext}} = 5.7 \text{kN for experiment 5} \]

From Figure 47 it is observed that a very good fit is achieved with this low value for the stiffness modulus of the asphalt concrete plate.
4.2.3.3 $p_{\text{last}} = 5.9 \text{ kN}$

For the last loadings of 5.9 kN more compaction of the sand underneath the asphalt concrete plate and some nonlinear (visco-elastic) behavior of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum displacements for the last loadings of 5.9 kN of experiment 5.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

Table 29: Measured maximum elastic deformation for $p_{\text{last}} = 5.9 \text{ kN}$ for experiment 5

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 1010 MPa
2. First sand layer: 46 MPa
3. Second sand layer: 46 MPa
4. Third sand layer: 46 MPa
5. Bottom layer: 68947.6 MPa

The asphalt concrete and sand layers have higher stiffness moduli than for the first loadings of 5.7 kN. The stiffness moduli tend to increase in time (during loading). Table 30 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>449.71</td>
<td>844.04</td>
<td>844.04</td>
<td>-445.63</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>432.41</td>
<td>-903.8</td>
<td>-903.8</td>
<td>1071.51</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>280.2</td>
<td>395.98</td>
<td>-68.03</td>
<td>-176.04</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>280.69</td>
<td>-375.36</td>
<td>136.94</td>
<td>160.51</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>144.49</td>
<td>127.25</td>
<td>-146.97</td>
<td>10.62</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>144.84</td>
<td>-100.51</td>
<td>157.08</td>
<td>-23.77</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>57.08</td>
<td>30.11</td>
<td>-57.82</td>
<td>14.92</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>57.18</td>
<td>-15.34</td>
<td>50.93</td>
<td>-18.92</td>
</tr>
</tbody>
</table>

Table 30: Results from calculations for $p_{\text{last}} = 5.9 \text{ kN}$ for experiment 5

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. As expected the maximum strains are smaller than for the loadings of 5.7 kN (indicated in bold in Table 30).
In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 48.

*Figure 48: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{\text{last}} = 5.9$ kN for experiment 5*

From Figure 48 it is observed that a good fit is achieved.
4.2.3.4 $p_{\text{first}} = 8.5 \text{ kN}$

For the first loadings of 8.5 kN some compaction of the sand underneath the asphalt concrete plate and nonlinear (visco-elastic) behavior of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum displacements for the first loadings of 8.5 kN of experiment 5.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement ($\mu$m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>620</td>
</tr>
<tr>
<td>V5</td>
<td>410</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>190</td>
</tr>
<tr>
<td>V1</td>
<td>60</td>
</tr>
<tr>
<td>V6</td>
<td>0</td>
</tr>
</tbody>
</table>

*Table 31: Measured maximum elastic deformation for $p_{\text{first}} = 8.5 \text{ kN}$ for experiment 5*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 1010 MPa
2. First sand layer: 48 MPa
3. Second sand layer: 48 MPa
4. Third sand layer: 48 MPa
5. Bottom layer: 68947.6 MPa

The asphalt concrete has the same layer stiffness modulus used for the previous loading of 5.9 kN. The sand layers have a slightly higher stiffness modulus, which can be expected for an increase of the load. Table 32 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement ($\mu$m)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>629.74</td>
<td>1197.26</td>
<td>1197.26</td>
<td>-621.81</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>604.75</td>
<td>-1281.05</td>
<td>-1281.05</td>
<td>1524.26</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>389.14</td>
<td>557.86</td>
<td>-103.55</td>
<td>-244.51</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>389.84</td>
<td>-527.26</td>
<td>200.93</td>
<td>222.26</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>199.25</td>
<td>177.52</td>
<td>-207.71</td>
<td>16.26</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>199.75</td>
<td>-138.85</td>
<td>221.69</td>
<td>-35.17</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>78.52</td>
<td>41.83</td>
<td>-80.08</td>
<td>20.6</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>78.67</td>
<td>-20.62</td>
<td>70.01</td>
<td>-26.28</td>
</tr>
</tbody>
</table>

*Table 32: Results from calculations for $p_{\text{first}} = 8.5 \text{ kN}$ for experiment 5*

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. The maximum strains are higher than for the first loadings of 5.7 kN (indicated in bold in Table 32). This is obvious since the load is higher.
In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 49.

**Figure 49: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{\text{final}} = 8.5$ kN for experiment 5**

From Figure 49 it is observed that a good fit is achieved.
4.2.3.5 $p_{\text{last}}= 8.7$ kN

For the last loadings of 8.7 kN more nonlinear behavior is expected to be observed. The table below gives an overview of the measured maximum displacements for the last loadings of 8.7 kN of experiment 5.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>610</td>
</tr>
<tr>
<td>V5</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>410</td>
</tr>
<tr>
<td>V2&amp;V4</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>195</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td></td>
<td>80</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
<tr>
<td></td>
<td>50</td>
</tr>
</tbody>
</table>

Table 33: Measured maximum elastic deformation for $p_{\text{last}}= 8.7$ kN for experiment 5

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

1. Asphalt concrete: 960 MPa
2. First sand layer: 49 MPa
3. Second sand layer: 52 MPa
4. Third sand layer: 52 MPa
5. Bottom layer: 68947.6 MPa

The asphalt concrete has a lower stiffness modulus than used for the previous loadings, which is expected if failure occurred. The first sand layer has a low stiffness while the other sand layers have the same but higher stiffness. This could be explained by compacting of the first sand layer and the presence of a space between the asphalt plate and the first layer (results in a combined stiffness modulus). Table 34 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>633.95</td>
<td>1248.55</td>
<td>1248.55</td>
<td>-625.76</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>606.85</td>
<td>-1337.16</td>
<td>-1337.16</td>
<td>1602.28</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>382.53</td>
<td>528.19</td>
<td>-54.96</td>
<td>-254.8</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>383.23</td>
<td>-526.66</td>
<td>123.28</td>
<td>251.09</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>190.64</td>
<td>176.15</td>
<td>-204.41</td>
<td>15.22</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>191.16</td>
<td>-157.8</td>
<td>218.21</td>
<td>-23.17</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>73.54</td>
<td>37.74</td>
<td>-88.03</td>
<td>27.08</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>73.69</td>
<td>-26.07</td>
<td>84.74</td>
<td>-31.1</td>
</tr>
</tbody>
</table>

Table 34: Results from calculations for $p_{\text{last}}= 8.7$ kN for experiment 5
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest (tensile) strains. The maximum strains are higher than the ones for the first loadings of 8.5 kN (indicated in bold in Table 34). The strains seem to decrease in time with repeated loading under the same load, except here where failure had occurred. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 50.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

*Figure 50: Measured elastic deformations vs. calculated elastic deformations from fitting $p_{\text{last}} = 8.7$ kN for experiment 5*

From Figure 50 it is observed that a reasonable fit is achieved.
4.2.4 Experiment 6

A total of 3 fits are made for experiment 6. A load with a magnitude of 7.5 kN was planned for the whole experiment (allowing for deviations), which is the reason why 3 points in time were chosen. These are after 10 pulses, 340 pulses and at the end of the test after 720 pulses. These 3 points in time are chosen because these represent the 3 stages where the measured deflections indicate significant changes in the profile as the experiment goes on.

![Figure 51: Vertical elastic displacements-experiment 6 [Deltares, 2013]](image)

Again it could be observed that the maximum elastic displacements measured decrease in time for the same load magnitude. During the first 60 pulses roughly equal elastic deformation profiles are measured (stage 1). After 340 pulses a significant change in the elastic displacement profile was measured. And after 350 pulses the profile remains the same until the last pulse.

It is observed by comparing the displacement measurements V4 and V5 that the asphalt concrete plate deforms asymmetrically around the position of the loading head. Deltares found that transducer V4 was placed on a thicker part of the asphalt concrete plate (about 3 mm thicker) compared to the thickness of the asphalt concrete plate at transducer V5 and at the thicker part of the asphalt concrete plate the sand surface underneath was more indented. Besides this larger thickness the higher elastic deformation measured by transducer V4 can also be caused by a locally stiffer part of the asphalt concrete (as this is known as an inhomogeneous material) and/or by locally relatively less compacted sand. In the worst case there could have been a problem with the measurement with transducer V4.
### 4.2.4.1 The first stage after 10 pulses

After 10 pulses a linear behavior for both the sand and the asphalt concrete is expected to be observed. The table below gives an overview of the measured maximum elastic displacements after 10 pulses during experiment 6.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>95</td>
</tr>
<tr>
<td>V4</td>
<td>100</td>
</tr>
<tr>
<td>V2</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

**Table 35: Measured maximum elastic deformation after 10 pulses during experiment 6**

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

6. Asphalt concrete: 3200 MPa
7. First sand layer: 45 MPa
8. Second sand layer: 45 MPa
9. Third sand layer: 45 MPa
10. Bottom layer: 68947.6 MPa

The asphalt concrete has a relatively lower stiffness than expected. It could be that the sand takes on a larger part of the loading. Table 36 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>389.39</td>
<td>487.28</td>
<td>487.28</td>
<td>-338.86</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>382.73</td>
<td>-526.4</td>
<td>-526.4</td>
<td>587.21</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>291.95</td>
<td>270.08</td>
<td>28.09</td>
<td>-160.55</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>292.12</td>
<td>-269.21</td>
<td>-0.2</td>
<td>154.77</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>183.36</td>
<td>113.1</td>
<td>-77.19</td>
<td>-19.33</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>183.49</td>
<td>-104.7</td>
<td>84.93</td>
<td>14.27</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>83.93</td>
<td>33.95</td>
<td>-56.75</td>
<td>12.28</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>83.99</td>
<td>-28.08</td>
<td>55.92</td>
<td>-14.34</td>
</tr>
</tbody>
</table>

**Table 36: Results from calculations for experiment 6 after 10 pulses**

From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest strains. The maximum strains are indicated in bold in Table 36.
In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 52.

**Figure 52: Measured elastic deformations vs. calculated elastic deformations from fitting experiment 6 after 10 pulses**

From Figure 52 it is observed that a good fit is achieved, if the measurement with transducer V4 is neglected. The calculated values are close to the measurements of the other transducers.
4.2.4.2 Second stage after 340 pulses

After 340 pulses the second stage of loading is marked where more compaction of the sand underneath the asphalt concrete plate and nonlinear (visco-elastic) behavior of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum elastic displacements after 340 pulses during experiment 3.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>95</td>
</tr>
<tr>
<td>V4</td>
<td>100</td>
</tr>
<tr>
<td>V2</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 37: Measured maximum elastic deformation after 340 pulses during experiment 3*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

6. Asphalt concrete: 3900 MPa  
7. First sand layer: 44 MPa  
8. Second sand layer: 44 MPa  
9. Third sand layer: 44 MPa  
10. Bottom layer: 68947.6 MPa

The asphalt concrete has a relatively higher stiffness than after 10 pulses, while the sand layers have a slightly lower stiffness modulus. The asphalt concrete responds with a higher stiffness to the loading after a longer loading time. Table 38 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>372.57</td>
<td>428.69</td>
<td>428.69</td>
<td>-307.1</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>367.06</td>
<td>-463.1</td>
<td>-463.1</td>
<td>513.53</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>286.8</td>
<td>243.91</td>
<td>36.06</td>
<td>-150.75</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>286.94</td>
<td>-244.32</td>
<td>-13.16</td>
<td>146.12</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>186.37</td>
<td>106.78</td>
<td>-63.59</td>
<td>-23.25</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>186.47</td>
<td>-100.32</td>
<td>70.37</td>
<td>19.16</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>88.77</td>
<td>33.94</td>
<td>-53.08</td>
<td>10.3</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>88.81</td>
<td>-29.19</td>
<td>52.77</td>
<td>-12.06</td>
</tr>
</tbody>
</table>

*Table 38: Results from calculations for experiment 6 after 340 pulses*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest strains. The maximum strains are indicated in bold in Table 38. It is noticed that these strains are less than those found for the fit after 10 pulses. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 53.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

*Figure 53: Measured elastic deformations vs. calculated elastic deformations from fitting experiment 6 after 340 pulses*

From Figure 53 it is observed that a good fit is achieved, if the measurement with transducer V4 is neglected. The calculated values are close to the measurements of the other transducers.
4.2.4.3 Final stage after 720 pulses

After 720 pulses the final stage of loading is marked where the most compaction of the sand underneath the asphalt concrete plate and nonlinear (visco-elastic) behavior of the asphalt concrete plate are expected to be observed. The table below gives an overview of the measured maximum elastic displacements for the last pulse of experiment 6.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>Maximum elastic displacement (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3</td>
<td>0</td>
</tr>
<tr>
<td>V5</td>
<td>95</td>
</tr>
<tr>
<td>V4</td>
<td>100</td>
</tr>
<tr>
<td>V2</td>
<td>200</td>
</tr>
<tr>
<td>V1</td>
<td>350</td>
</tr>
<tr>
<td>V6</td>
<td>418</td>
</tr>
</tbody>
</table>

*Table 39: Measured maximum elastic deformation for the last pulse of experiment 6*

After going through the fitting procedure the following stiffness (layer modulus) is found for the input of each layer:

6. Asphalt concrete: 3600 MPa
7. First sand layer: 45 MPa
8. Second sand layer: 45 MPa
9. Third sand layer: 45 MPa
10. Bottom layer: 68947.6 MPa

Compared to the elastic deflection profile after 340 pulses the asphalt concrete has a relatively lower stiffness modulus for the last pulse (which is expected in case of failure) and the sand layers have the same magnitude for the stiffness modulus for all 3 stages. Table 40 gives an overview of the results from calculations with the above mentioned fit.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>Vertical elastic displacement (μm)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
<th>Normal MicroStrain (z-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>353.27</td>
<td>422.77</td>
<td>422.77</td>
<td>-298.86</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>347.68</td>
<td>-456.72</td>
<td>-456.72</td>
<td>507.79</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>268.7</td>
<td>237.68</td>
<td>30.37</td>
<td>-144.33</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>268.85</td>
<td>-237.56</td>
<td>-7.05</td>
<td>139.56</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>171.9</td>
<td>101.95</td>
<td>-64.8</td>
<td>-20</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>172</td>
<td>-95.16</td>
<td>71.51</td>
<td>15.8</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>80.36</td>
<td>31.55</td>
<td>-50.96</td>
<td>10.45</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>80.4</td>
<td>-26.68</td>
<td>50.47</td>
<td>-12.21</td>
</tr>
</tbody>
</table>

*Table 40: Results from calculations for experiment 6 after 720 pulses*
From the results it is clear that the part of the asphalt concrete plate right underneath the loading head exerts the largest strains. The maximum strains are indicated in bold in Table 40. In order to compare the measured maximum elastic deflections with the calculated fit the values are plotted and illustrated in Figure 54.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

*Figure 54: Measured elastic deformations vs. calculated elastic deformations from fitting experiment 6 after 720 pulses*

From Figure 54 it is observed that a good fit is achieved, if the measurement with transducer V4 is neglected. The calculated values are close to the measurements of the other transducers.
4.3 The stiffness modulus of the asphalt mixes

The stiffness and strength of asphalt mixes can be determined with the help of nomograms. Some properties of an asphalt mixture must be known and can also be measured by performing tests. These tests have been performed by Deltares and KOAC-NPC.

The visco-elastic properties of the bitumen cause asphalt mixtures to have a visco-elastic behavior. This is why bitumen and also asphalt mixtures have no Young's modulus ($E_{\text{mix}}$), but a stiffness modulus ($S_{\text{mix}}$) which needs to be defined. In formula:

$$S_{\text{mix}} = \frac{\sigma}{\varepsilon_{t,T}}$$

Where: $\sigma$= applied bending stress (MPa)

$\varepsilon_{t,T}$= strain as a function of loading time $t$ and temperature $T$

4.3.1 Stiffness modulus of the new asphalt concrete

In this paragraph the stiffness modulus of the asphalt mix used in experiments 2, 3 and 4 is determined using nomograms. Also an estimation of the fatigue curve is determined.

4.3.1.1 Input data new asphalt

The available data of the new asphalt concrete mix is listed below [KOAC-NPC, 2012]:

- PEN (in $0.1\text{mm}$)= 77
- Penetration Index= -1.0
- Softening point= 46.8°C
- Volume percentage of aggregate ($V_g$)= 85.40% [%]
- Volume percentage of bitumen ($V_b$)= 14.30% [%]

The percentages bitumen, aggregate and hollow space were measured from specimens taken for the four-point bend test beams.

According to the 2 four-point bending tests executed the stiffness modulus of the asphalt mixture at 10°C for a 10 Hz loading should be between 16015 MPa (for 5°C) and 4679 MPa (for 20°C). The results of these tests are included in APPENDIX A.

4.3.1.2 Determining the stiffness modulus and fatigue curve of the new asphalt mix

Before determining the value of the stiffness modulus of the asphalt mix ($S_{\text{mix}}$), the stiffness modulus of the bitumen ($S_{\text{bit}}$) should be determined. For an estimation of the stiffness modulus of the bitumen the Van der Poel nomogram will be used. The input for determining the stiffness modulus with this nomogram is:

- Time of loading= 0.1 s
- $\Delta T$= 46,8-10= 36.8°C
- Penetration Index= -1.0

The stiffness modulus of the bitumen than reads: $S_{\text{bit}}= 5\times10^7 \text{ N/m}^2 = 50 \text{ MPa}$ (see the blue line in the nomogram, Figure 55).
Figure 55: Van der Poel nomogram for determining the stiffness modulus of bitumen for the new asphalt concrete.
The stiffness modulus of the asphalt mix \( (S_{\text{mix}}) \) is dependent on the volumetric composition of the asphalt mixture and of the bitumen properties \( (S_{\text{bit}}) \). The stiffness modulus can be determined using the Ugé (Shell) nomogram when using conventional, pure bitumen.

The following input is needed:

- volume percentage of aggregate: \( V_g = 85.40\% \)
- volume percentage of bitumen: \( V_b = 14.30\% \)
- the stiffness modulus of the bitumen: \( S_{\text{bit}} = 5*10^7 \text{ N/m}^2 \)

The stiffness modulus of the asphalt mixture then reads: \( S_{\text{mix}} = 7.5*10^9 \text{ N/m}^2 = 7500 \text{ MPa} \) (see the blue line in the nomogram, Figure 56).

This estimation is lower than 8500 MPa. The expected value was between 8500 MPa (5 Hz, 10°C) and 15200 MPa (10 Hz, 5°C). When back-calculating the last loadings of 12.4 kN of experiment 4 (see 4.3.2.6) stiffness moduli of 7000 MPa and 6800 were found, which is in the order of magnitude of the determined stiffness. For experiment 3 after 51 hours the highest stiffness of 5300 MPa was back-calculated (see 4.3.1.3).

However these values are in between the values of the 2 four-point bend tests executed. The stiffness modulus of the asphalt mixture at 10°C for a 10 Hz loading is between 16015 MPa (for 5°C) and 4679 MPa (for 20°C).
Figure 56: Ugé (Shell) nomogram for determining the stiffness modulus of the new asphalt concrete mix [CT2800]
With the Shell nomogram for determining the (linear on a double log scale) fatigue curves for asphalt mixes with bitumen the strain for $10^4$ and $10^8$ fatigue life cycles will be determined (the blue lines, Figure 57). The four-point bend tests were performed with a constant strain of 50 μm/mm [KOAC-NPC, 2012].

![Shell nomogram for determining the fatigue curves for the new asphalt concrete](CT2800)

**Figure 57: Shell nomogram for determining the fatigue curves for the new asphalt concrete [CT2800]**

This reads:
- An initial strain of $6.2 \times 10^{-4}$ for $10^4$ fatigue life cycles.
- An initial strain of $9 \times 10^{-5}$ for $10^8$ fatigue life cycles.

With these values a fatigue curve can be constructed. It would be linear, which would be good enough for asphalt on roads, but in this case of asphalt revetments a curved fatigue curve is preferred. But for a rough estimation the function for a linear fatigue curve will be calculated in the form:

$$\log N = a + b \log \varepsilon$$

This gives:
- $\log 10^4 = a + b \log (6.2 \times 10^{-4})$
- $\log 10^8 = a + b \log (9 \times 10^{-5})$

And yields: $a = -11.308$ and $b = -4.772$. The fatigue curve: $\log N = -11.308 - 4.772 \log \varepsilon$
4.3.2 Stiffness modulus of the old asphalt concrete

In this paragraph the stiffness modulus of the asphalt mix used in experiments 5 and 6 is determined with nomograms. Also an estimation of the fatigue curve is determined.

4.3.2.1 Input data old asphalt

The available data of the old asphalt concrete mix is listed below [KOAC-NPC, 2013]:

- PEN (in 0.1 mm) = 20.5
- Penetration Index = -0.25
- Softening point = 63.3°C
- Specific gravity of compacted asphalt mix ($\rho_{mix}$) = 2.403 kg/m$^3$
- Percentage of bitumen = (6.9+7.2)/2 = 7.05% [\%/m]

These values are averaged since these are measured for 2 specimens (1 on top and 1 on the underside of the original specimen). The values are included in APPENDIX A.

4.3.2.2 Determining the stiffness modulus and fatigue curve of the old asphalt mix

Before determining the value of the stiffness modulus of the asphalt mix ($S_{mix}$), the stiffness modulus of the bitumen ($S_{bit}$) should be determined. For an estimation of the stiffness modulus of the bitumen the van der Poel nomogram is used. The input for determining the stiffness modulus with this nomogram is:

- Time of loading = 0.1 s
- $\Delta T$ = 63.3-10 = 53.3°C
- Penetration Index = -0.25

The stiffness modulus of the bitumen then reads: $S_{bit} = 2*10^8$ N/m$^2$ = 200 MPa (see the blue line in the nomogram, Figure 58). This is a higher value than the stiffness modulus of the bitumen used in the new asphalt concrete. This outcome is expected, because the bitumen becomes stiffer due to ageing.
Figure 58: Van der Poel nomogram for determining the stiffness modulus of bitumen for the old asphalt concrete.

To determine the stiffness modulus of a 50 pen bitumen at the test conditions, connect 0.02s on the loading time scale with (53.5 + 1.5 – 5 = 50°C) on the temperature scale. Stiffness modulus is 1.5 x 10^6 Pa at a PI of 0.

Conditions:
- Loading time: 0.02 sec
- Temperature: 5°C
- Bitumen properties:
  - Penetration at 25°C: 50 mm
  - Softening point (°F): 53.5°C
  - PI: 0.0
The stiffness modulus of the asphalt mix ($S_{\text{mix}}$) is dependent on the volumetric composition of the asphalt mixture and of the bitumen properties ($S_{\text{bit}}$). The stiffness modulus can be determined using the Ugé (Shell) nomogram when using conventional, pure bitumen.

The percentages of aggregate and bitumen are calculated as follows (method used in the course CT3041, TU Delft):

Specific gravity aggregate: $\rho_{\text{aggregate}} = 2.65 \, \text{kg/m}^3$

Specific gravity bitumen: $\rho_{\text{bitumen}} = 1.03 \, \text{kg/m}^3$

Percentage of bitumen in mix: $(6.9+7.2)/2 = 7.05\% \, [\text{m/m}]$

Specific gravity of compacted asphalt mix: $\rho_{\text{mix}} = (2.268+2.197)/2 = 2.233 \, \text{kg/m}^3$

Maximum specific gravity of asphalt mix:

$$\rho_{\text{mm}} = 100/((100-\%\text{bitumen})/\rho_{\text{aggregate}})+(%\text{bitumen}/\rho_{\text{bitumen}}) = 2.385 \, \text{kg/m}^3$$

Specific gravity of the compacted aggregate: $\rho_{\text{sc}} = \rho_{\text{mix}}(100-%\text{bitumen})/100 = 2.076 \, \text{kg/m}^3$

Volume percentage of aggregate: $V_{\text{g}} = 100*(\rho_{\text{sc}}/\rho_{\text{aggregate}}) = 78.34\% \, [\text{v/v}]$

Volume percentage of bitumen: $V_{\text{b}} = 100*(\rho_{\text{mix}}/\rho_{\text{mm}})-V_{\text{g}} = 15.29\% \, [\text{v/v}]$

Volume percentage of air: $V_{\text{a}} = 100*(1-(\rho_{\text{mix}}/\rho_{\text{mm}})) = 6.37\% \, [\text{v/v}]$

Summary of the needed input values:

- volume percentage of aggregate: $V_{\text{g}} = 78.34\% \, [\text{v/v}]$
- volume percentage of bitumen: $V_{\text{b}} = 15.29\% \, [\text{v/v}]$
- the stiffness modulus of the bitumen: $S_{\text{bit}} = 2*10^8 \, \text{N/m}^2$

The stiffness modulus of the asphalt mixture then reads: $S_{\text{mix}} = 9*10^9 \, \text{N/m}^2 = 9000 \, \text{MPa}$ (see the blue line in the nomogram, Figure 59). Back-calculation of experiment 6 after 340 pulses (see 4.3.4.2) indicates the highest stiffness modulus for the asphalt concrete mix of 3900 MPa. This is still a factor 2.3 below the determined stiffness modulus. However the asphalt concrete plates used in experiment 5 and 6 did fail.
Figure 59: Ugé (Shell) nomogram for determining the stiffness modulus of the old asphalt concrete mix [CT2800]
With the Shell nomogram for determining the (linear on double log scale) fatigue curves for asphalt mixes with bitumen the strain for $10^4$ and $10^8$ fatigue life cycles will be determined (the blue lines, Figure 60). The four-point bend tests were performed with a constant strain of 50 $\mu$m/mm (KOAC-NPC, 2012).

**Figure 60: Shell nomogram for determining the fatigue curves for the old asphalt concrete [CT2800]**

This reads:
Initial strain of $7\times10^{-4}$ for $10^4$ fatigue life cycles.
Initial strain of $1\times10^{-4}$ for $10^8$ fatigue life cycles.

With these figures fatigue curves can be constructed. These would be linear, which would be good enough for asphalt on roads, but in this case of asphalt revetments a curved fatigue curve is preferred. But for a rough estimation the function for a linear fatigue curve will be calculated in the form:

$$\log N = a + b \log \varepsilon$$

This gives:
$$\log 10^4 = a + b \log (7\times10^{-4})$$
$$\log 10^8 = a + b \log (1\times10^{-4})$$

And yields: $a = -10.933$ and $b = -4.733$. The fatigue curve: $\log N = -10.933 - 4.733 \log \varepsilon$
4.3.3 Fatigue curves

The fatigue curves are now determined and illustrated in figure 61.

![Linear fatigue curves](image)

**Figure 61: The calculated linear fatigue curves for the new and old asphalt concrete**

The calculated linear fatigue curves for the new and old asphalt concrete do not differ much, because the gradients (b) and the value of a for both lines are almost of the same magnitude. Deltares has determined the nonlinear fatigue curves for calculating the moment of failure with a better accuracy. For example the nonlinear fatigue curve for the old asphalt concrete used for experiment 6 is given in figure 62.

![Nonlinear fatigue curve](image)

**Figure 62: Nonlinear fatigue curve for the old asphalt concrete in experiment 6 [Deltares, 2013, B]**

It can be observed that the linear fatigue curve comes fairly close to the nonlinear curves for strains around $10^3 \mu m/m$. For strains lower than $10^3 \mu m/m$ the linear fatigue curve is steeper than for the nonlinear fatigue curves, which means that for these cases the number of fatigue life cycles is overestimated with the linear fatigue curves. In the next paragraph the linear fatigue curves will be used to get a rough idea of the results with WESLEA.
4.4 Evaluation

The Miner sums are calculated for the asphalt concrete plates of experiment 3, 4, 5 and 6 with the determined linear fatigue curves and the known number of loadings \((n_i)\) for each loading case. The number of loads leading to failure due to fatigue \((N_{f,i})\) is calculated from filling in the maximum (tensile) strains in the \(x\)- and \(y\)-direction into the equation found for the fatigue curves in paragraph 4.3.

4.4.1 Experiment 3

For experiment 3 new asphalt concrete produced in a laboratory environment was used. The total Miner sums are given in Table 41 for 10 minutes, 10 hours and 51 hours after the start of the experiment.

<table>
<thead>
<tr>
<th>New asphalt concrete</th>
<th>(t=10) minutes</th>
<th>(t=10) hours</th>
<th>(t=51) hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\varepsilon) (m/m)</td>
<td>0.0012512</td>
<td>0.0010285</td>
<td>0.00064943</td>
</tr>
<tr>
<td>(N_{f,i}) (-)</td>
<td>351</td>
<td>893</td>
<td>8015</td>
</tr>
<tr>
<td>(N_i) (-)</td>
<td>60</td>
<td>3540</td>
<td>15120</td>
</tr>
<tr>
<td>Miner sum (-)</td>
<td>0.171</td>
<td>4.134</td>
<td>6.021</td>
</tr>
</tbody>
</table>

*Table 41: Total Miner sums at \(t=10\) min, 10 hours and 51 hours during experiment 3*

From the total Miner sums it is expected that the asphalt concrete plate would fail due to fatigue after approximately 2.2 hours of 12.2 kN loading (800 loadings after the start of the experiment). No signs of failure in the sense of cracks were observed after the experiment. There is a possibility that if there was a crack after approximately 2.2 hours, the crack could have been repaired by self-healing of the asphalt concrete due to the flatter elastic displacement profile. The maximum elastic strains occurring at each stage are given in figure 63.

![Calculated maximum strains for experiment 3](image)

*Figure 63: Calculated maximum strains for experiment 3*

It is observed that the maximum elastic strains decrease in time under loading. The permanent deformations and therefore also the permanent strains increase in time under loading as shown in figure 64.
The increase of the permanent displacements can partly be explained by the compaction of the sand. That compaction occurs, can be indicated by the higher relative density measured at the end of the experiment (see table 42).

The measured relative density is larger for the top layers than the layers beneath, which is also expected to occur. Unfortunately the accuracy of the measured relative density is too large to be certain that the relative density increased during the experiment.
4.4.2 Experiment 4

New asphalt concrete was used for experiment 4, produced in a laboratory environment. The Miner sums for the back-calculations with and without V5 are given in Table 43 and Table 44 for the first and last measurements of loading stages 2.25 kN, 4.3 kN and 12.4 kN. The number of loadings is given (by Deltares) as a total for each load and is therefore equally split for the first and last loadings to calculate the Miner sums for each loading phase.

<table>
<thead>
<tr>
<th>New asphalt concrete</th>
<th>( P_{\text{first}} = 2.25 \text{ kN} )</th>
<th>( P_{\text{last}} = 2.25 \text{ kN} )</th>
<th>( P_{\text{first}} = 4.3 \text{ kN} )</th>
<th>( P_{\text{last}} = 4.3 \text{ kN} )</th>
<th>( P_{\text{first}} = 12.4 \text{ kN} )</th>
<th>( P_{\text{last}} = 12.4 \text{ kN} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \varepsilon ) (m/m)</td>
<td>0.0002388</td>
<td>0.0001474</td>
<td>0.0003231</td>
<td>0.0002490</td>
<td>0.0007557</td>
<td>0.0004685</td>
</tr>
<tr>
<td>( N_{ti} ) (-)</td>
<td>949104</td>
<td>9488642</td>
<td>224281</td>
<td>778417</td>
<td>3888</td>
<td>38093</td>
</tr>
<tr>
<td>Miner sum (-)</td>
<td>180</td>
<td>180</td>
<td>486</td>
<td>486</td>
<td>720</td>
<td>240</td>
</tr>
<tr>
<td></td>
<td>0.00019</td>
<td>0.000019</td>
<td>0.002167</td>
<td>0.00062</td>
<td>0.185</td>
<td>0.0063</td>
</tr>
</tbody>
</table>

*Table 43: Miner sums for the loading stages 2.25 kN, 4.3 kN and 12.4 kN of experiment 4 with V5*

The maximum elastic strains occurring at each stage are given in figure 65 to obtain a better view of these results.

![Calculated maximum strains for experiment 4 with V5](image)

*Figure 65: Calculated maximum strains for experiment 4 including V5*

Also here it can be observed that the maximum elastic strains decrease in time under the same loading. The maximum strains increases if a higher load is applied. The back-calculated strains occurring for a loading of 12.4 kN are smaller than the back-calculated strains occurring for a loading of 12.2 kN in experiment 3. The sand bed underneath the asphalt concrete plate had already have the chance to get compacted under the loadings of 2.25 kN and 4.3 kN, resulting in smaller occurring strains for a loading of 12.4 kN during experiment 4.
The same data is given for the calculation excluding the measurements with transducer V5.

<table>
<thead>
<tr>
<th>New asphalt concrete</th>
<th>p(_{\text{first}}) = 2.25 kN</th>
<th>p(_{\text{last}}) = 2.25 kN</th>
<th>p(_{\text{first}}) = 4.3 kN</th>
<th>p(_{\text{last}}) = 4.3 kN</th>
<th>p(_{\text{first}}) = 12.4 kN</th>
<th>p(_{\text{last}}) = 12.4 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\varepsilon) (m/m)</td>
<td>0.0002388</td>
<td>0.0001387</td>
<td>0.0002791</td>
<td>0.0003169</td>
<td>0.0007537</td>
<td>0.0004813</td>
</tr>
<tr>
<td>(N_{ii}) (-)</td>
<td>949104</td>
<td>12675930</td>
<td>451023</td>
<td>246230</td>
<td>3939</td>
<td>33478</td>
</tr>
<tr>
<td>(N_i) (-)</td>
<td>180</td>
<td>180</td>
<td>486</td>
<td>486</td>
<td>720</td>
<td>240</td>
</tr>
<tr>
<td>Miner sum (-)</td>
<td>0.00019</td>
<td>0.000014</td>
<td>0.00107</td>
<td>0.00197</td>
<td>0.183</td>
<td>0.00717</td>
</tr>
</tbody>
</table>

*Table 44: Miner sums for the loading stages 2.25 kN, 4.3 kN and 12.4 kN of experiment 4 without V5*

Up till the last loadings of 12.4 kN no failure due to fatigue is expected, because for both cases the total Miner sum is less than 1 (approximately 0.19 for both). The loadings were increased to 16.25 kN and 19.85 kN (back-calculations were not possible with WESLEA), so there is a possibility that failure due to fatigue could occur. No signs of failure in the sense of cracks were observed afterwards. Either the asphalt plate did not fail or the cracks were not visible.

The maximum elastic strains occurring at each stage are given in figure 66 to obtain a better view of these results.

![Calculated maximum strains for experiment 4 excluding V5](image)

*Figure 66: Calculated maximum strains for experiment 4 excluding V5*

It can be observed that the maximum elastic strains decrease in time under the loadings of 2.25 kN and 12.4 kN, but slightly increased for a loading of 4.3 kN (possibly due to less accuracy by excluding V5). The maximum strains increases if a higher load is applied.
The permanent deformations and therefore also the permanent strains increase in time under loading as shown in figure 67.

![Figure 67: Displacement measurements during experiment 4 [Deltares, 2012]](image)

The increase of the permanent displacements can partly be explained by the compaction of the sand. That compaction occurs, can be indicated by the higher relative density measured at the end of the experiment (see table 45).

<table>
<thead>
<tr>
<th>Experiment 4</th>
<th>Average at sides (%)</th>
<th>At center (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prepared</td>
<td>63 ± 2.5</td>
<td>63 ± 2.5</td>
</tr>
<tr>
<td>top layer</td>
<td>72.98 ± 8</td>
<td>67.41 ± 8</td>
</tr>
<tr>
<td>100 mm below surface of sand bed</td>
<td>67.45 ± 8</td>
<td>64.8 ± 8</td>
</tr>
<tr>
<td>200 mm below surface of sand bed</td>
<td>65.32 ± 8</td>
<td>59.05 ± 8</td>
</tr>
</tbody>
</table>

*Table 45: Relative density at the end of experiment 4 [Deltares, 2012]*

The measured relative density is larger for the top layers than the layers beneath, which is also expected to occur. Unfortunately the accuracy of the measured relative density is too large to be certain that the relative density increased during the experiment.
4.4.3 Experiment 5

Old asphalt concrete taken from an asphalt concrete dike revetment from the seaport Groningen was used for experiment 5. The Miner sums are given in Table 46 for the loading stages 2.4 kN, 5.7 kN, 5.9 kN, 8.5 kN and 8.7 kN. The number of loadings is given (by Deltares) as a total for each load and is therefore equally split for the first and last loadings to calculate the Miner sums for each loading phase.

\[
\begin{array}{|c|c|c|c|c|c|}
\hline
\text{Old asphalt concrete} & p_{\text{last}}= 2.4 \text{ kN} & p_{\text{first}}= 5.7 \text{ kN} & p_{\text{last}}= 5.9 \text{ kN} & p_{\text{first}}= 8.5 \text{ kN} & p_{\text{last}}= 8.7 \text{ kN} \\
\varepsilon (\text{m/m}) & 0.0004746 & 0.001098 & 0.0009038 & 0.00128105 & 0.00133716 \\
N_{ij} (-) & 62925 & 1189 & 2984 & 572 & 467 \\
N_i (-) & 419 & 137 & 138 & 293 & 293 \\
\text{Miner sum (-)} & 0.007 & 0.115 & 0.046 & 0.512 & 0.627 \\
\hline
\end{array}
\]

Table 46: Miner sums for the loading stages 2.4 kN, 5.7 kN, 5.9 kN, 8.5 kN and 8.7 kN of experiment 5

The total of all the Miner sums for each of the load steps is equal to 1.31. This sum is calculated excluding the maximum strains of the deflections of first loads of 2.1 kN. In order to make up for this the total number of loadings is assigned to the deflection profile of 2.4 kN. The total Miner sum is larger than 1, which indicates that failure due to fatigue is expected approximately 1500 s (150 loadings) before the last pulse of the experiment. After experiment 5 cracks were detected in the area under the loading head, which confirm failure due to fatigue. The maximum elastic strains occurring at each stage are presented in figure 68.

It is observed that the maximum elastic strains decrease in time under the loading of 5.7 kN to 5.9 kN, but slightly increased for a loading of 8.5 kN to 8.7 kN (possibly due to the fact that failure occurred). It is also observed that the maximum strains increases if a higher load is
applied. The permanent deformations and therefore also the permanent strains increase in time under loading as shown in figure 69.

![Figure 69: Displacement measurements during experiment 5 [Deltares, 2013]](image)

The increase of the permanent displacements can partly be explained by the compaction of the sand. That compaction occurs, can be indicated by the higher relative density measured at the end of the experiment (see table 47).

<table>
<thead>
<tr>
<th>Experiment 5</th>
<th>Average at sides (%)</th>
<th>At center (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prepared</td>
<td>67.3 ± 2.5</td>
<td>67.3 ± 2.5</td>
</tr>
<tr>
<td>top layer</td>
<td>74.48 ± 8</td>
<td>74.22 ± 8</td>
</tr>
<tr>
<td>70 mm below surface of sand bed</td>
<td>-</td>
<td>75.18 ± 8</td>
</tr>
<tr>
<td>140 mm below surface of sand bed</td>
<td>65.61 ± 8</td>
<td>70.69 ± 8</td>
</tr>
<tr>
<td>210 mm below surface of sand bed</td>
<td>-</td>
<td>73.49 ± 8</td>
</tr>
<tr>
<td>280 mm below surface of sand bed</td>
<td>66.26 ± 8</td>
<td>73.04 ± 8</td>
</tr>
</tbody>
</table>

*Table 47: Relative density at the end of experiment 5 [Deltares, 2013]*

The measured relative density is larger for the top layers than the layers beneath, which is also expected to occur. Unfortunately the accuracy of the measured relative density is too large to be certain that the relative density increased during the experiment.
4.4.4 Experiment 6

For experiment 6 old asphalt concrete taken from an asphalt concrete dike revetment from the seaport Groningen was used. The Miner sums are given in Table 48 for 10, 340 and 720 pulses after the start of the experiment.

<table>
<thead>
<tr>
<th>New asphalt concrete</th>
<th>After 10 pulses</th>
<th>After 340 pulses</th>
<th>After 720 pulses</th>
</tr>
</thead>
<tbody>
<tr>
<td>ε (m/m)</td>
<td>0.0005264</td>
<td>0.0004631</td>
<td>0.00045672</td>
</tr>
<tr>
<td>N_f,j (-)</td>
<td>38537</td>
<td>70671</td>
<td>75467</td>
</tr>
<tr>
<td>N_i (-)</td>
<td>10</td>
<td>330</td>
<td>380</td>
</tr>
<tr>
<td>Miner sum (-)</td>
<td>0.0003</td>
<td>0.00467</td>
<td>0.00504</td>
</tr>
</tbody>
</table>

Table 48: Total Miner sums after 10, 340 and 720 pulses during experiment 6

From the total Miner sum of 0.01 it is not expected that the asphalt concrete plate would fail due to fatigue. However cracks were visible after the experiment was performed. This outcome was certainly not expected since the calculations carried out by KOAC-NPC predicted failure due to fatigue. The maximum elastic strains are in the order of 500 μm/m, which gets a larger number fatigue life cycles assigned to it with the linear fatigue curve than with the nonlinear fatigue curve. This case is a clear example for which nonlinear curves would have a better accuracy in determining the number of fatigue life cycles. The maximum elastic strains occurring at each stage are presented in figure 70.

![Calculated maximum strains for experiment 6](image)

Figure 70: Calculated maximum strains for experiment 6

It is observed that the maximum elastic strains decrease in time under a loading of 7.5 kN to 7.1 kN. The permanent deformations and therefore also the permanent strains increase in time under loading as shown in figure 71.
The increase of the permanent displacements can partly be explained by the compaction of the sand. That compaction occurs, can be indicated by the higher relative density measured at the end of the experiment (see table 49).

<table>
<thead>
<tr>
<th>Experiment 6</th>
<th>Average at sides (%)</th>
<th>At center (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prepared</td>
<td>65.9 ± 2.5</td>
<td>65.9 ± 2.5</td>
</tr>
<tr>
<td>top layer</td>
<td>79.33 ± 8</td>
<td>80.11 ± 8</td>
</tr>
<tr>
<td>70 mm below surface of sandbed</td>
<td>-</td>
<td>72.74 ± 8</td>
</tr>
<tr>
<td>140 mm below surface of sandbed</td>
<td>70.49 ± 8</td>
<td>71.26 ± 8</td>
</tr>
<tr>
<td>210 mm below surface of sandbed</td>
<td>-</td>
<td>71.04 ± 8</td>
</tr>
<tr>
<td>280 mm below surface of sandbed</td>
<td>60.47 ± 8</td>
<td>66.35 ± 8</td>
</tr>
</tbody>
</table>

**Table 49: Relative density at the end of experiment 6 [Deltares, 2013, B]**  

The measured relative density is larger for the top layers than the layers beneath, which is also expected to occur. Unfortunately the accuracy of the measured relative density is too large to be certain that the relative density increased during the experiment, except for the top layer. The relative density of the top layer certainly increased during the experiment.
4.4.5 Overview of the layer stiffness moduli

In this paragraph an overview of the stiffness moduli attained from the back-calculations is given for each stage of each experiment. For the back-calculations with the WESLEA software the focus was on fitting the profile in order to get accurate values for the maximum strains rather than attaining a reasonable stiffness modulus for asphalt concrete. Also findings regarding the stiffness moduli of research conducted by Deltares and KOAC-NPC will be presented, to make a comparison of the findings with the back-calculations.

In experiment 3 (see table 50) compaction of the upper sand layers is observed after 10 minutes as the stiffness modulus for the first sand layer is higher than for the second and third sand layers. After 10 and 51 hours of loading a lower stiffness modulus of the upper sand layer(-s) is found. The first sand layer has a lower stiffness modulus possibly because some space formed right underneath the asphalt plate below the loading head for which a combined stiffness modulus is found caused by compaction and the increasing permanent deformation. It is observed that the asphalt stiffness modulus increases in time (see figure 72).

<table>
<thead>
<tr>
<th>Input</th>
<th>Layer thickness (mm)</th>
<th>t=10 minutes</th>
<th>t=10 hours</th>
<th>t= 51 hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC (MPa)</td>
<td>50</td>
<td>1440</td>
<td>2700</td>
<td>5300</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>100</td>
<td>82</td>
<td>30</td>
<td>21</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>200</td>
<td>42</td>
<td>60</td>
<td>40</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>500</td>
<td>40</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Other (MPa)</td>
<td>∞</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
</tr>
</tbody>
</table>

Table 50 Overview of the stiffness moduli of experiment 3

Figure 72: Calculated stiffness modulus of the asphalt concrete plate used in experiment 3
Also for experiment 4 it is observed that the asphalt stiffness modulus increases in time (see tables 51 and 52 and figures 73 and 74), except for the loads of 4.3 kN (fit without V5). The asphalt stiffness modulus increases if the load is increased. The sand stiffness moduli are of the same magnitude in both cases of fitting, except for $p_{\text{first}} = 2.25$ kN and $p_{\text{first}} = 12.4$ kN (second sand layer, calculations including the measurements with V5).

<table>
<thead>
<tr>
<th>Input with V5</th>
<th>Layer thickness (mm)</th>
<th>$p_{\text{first}} = 2.25$ kN</th>
<th>$p_{\text{last}} = 2.25$ kN</th>
<th>$p_{\text{first}} = 4.3$ kN</th>
<th>$p_{\text{last}} = 4.3$ kN</th>
<th>$p_{\text{first}} = 12.4$ kN</th>
<th>$p_{\text{last}} = 12.4$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC (MPa)</td>
<td>50</td>
<td>1550</td>
<td>3300</td>
<td>2800</td>
<td>4000</td>
<td>3600</td>
<td>7000</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>100</td>
<td>60</td>
<td>54</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>200</td>
<td>48</td>
<td>52</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>500</td>
<td>48</td>
<td>52</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Other (MPa)</td>
<td>$\infty$</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
</tr>
</tbody>
</table>

*Table 51 Overview of the stiffness moduli of experiment 4 with V5*

**Figure 73: Calculated stiffness modulus of the asphalt concrete plate used in experiment 4 including V5**

![Asphalt stiffness modulus for experiment 4 with V5](image)

<table>
<thead>
<tr>
<th>Input without V5</th>
<th>Layer thickness (mm)</th>
<th>$p_{\text{last}} = 2.25$ kN</th>
<th>$p_{\text{first}} = 4.3$ kN</th>
<th>$p_{\text{last}} = 4.3$ kN</th>
<th>$p_{\text{first}} = 12.4$ kN</th>
<th>$p_{\text{last}} = 12.4$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC (MPa)</td>
<td>50</td>
<td>3700</td>
<td>3700</td>
<td>2900</td>
<td>3800</td>
<td>6800</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>100</td>
<td>49</td>
<td>40</td>
<td>49</td>
<td>48</td>
<td>49</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>200</td>
<td>49</td>
<td>40</td>
<td>49</td>
<td>48</td>
<td>49</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>500</td>
<td>49</td>
<td>40</td>
<td>49</td>
<td>48</td>
<td>49</td>
</tr>
<tr>
<td>Other (MPa)</td>
<td>$\infty$</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
</tr>
</tbody>
</table>

*Table 52 Overview of the stiffness moduli of experiment 4 without V5*
Figure 74: Calculated stiffness modulus of the asphalt concrete plate used in experiment 4 excluding V5
When looking at experiment 5 (table 53) the stiffness moduli of both the sand and the asphalt concrete increase with increased loading and the loading time, except for the last loading of 2.4 kN and 8.7 kN. The loadings of 2.1 and 2.4 kN required very low asphalt concrete stiffness moduli. It was not even possible to fit the loading of 2.1 kN properly.

<table>
<thead>
<tr>
<th>Input</th>
<th>Layer thickness (mm)</th>
<th>$P_{last} = 2.4$ kN</th>
<th>$P_{first} = 5.7$ kN</th>
<th>$P_{last} = 5.9$ kN</th>
<th>$P_{first} = 8.5$ kN</th>
<th>$P_{last} = 8.7$ kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC (MPa)</td>
<td>50</td>
<td>580</td>
<td>800</td>
<td>1010</td>
<td>1010</td>
<td>960</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>100</td>
<td>53</td>
<td>37</td>
<td>46</td>
<td>48</td>
<td>49</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>200</td>
<td>53</td>
<td>36</td>
<td>46</td>
<td>48</td>
<td>52</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>500</td>
<td>50</td>
<td>36</td>
<td>46</td>
<td>48</td>
<td>52</td>
</tr>
<tr>
<td>Other (MPa)</td>
<td>$\infty$</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
</tr>
</tbody>
</table>

*Table 53 Overview of the stiffness moduli of experiment 5*

The last loading of 8.7 kN had a slightly lower value for the asphalt stiffness modulus than for the first loadings of 8.5 kN, which could be an indication that failure did occur (see figure 75).

![Asphalt stiffness modulus for experiment 5](image)

*Figure 75: Calculated stiffness modulus of the asphalt concrete plate used in experiment 5*
An overview of the stiffness moduli for the 3 loading stages back-calculated for experiment 6 is given in table 54. The stiffness modulus of the asphalt concrete increases from the first to the second stage and decreases for the third stage (see figure 76). This could indicate failure due to fatigue. The stiffness moduli for the sand layers are of the same magnitude for all of the stages.

<table>
<thead>
<tr>
<th>Input</th>
<th>Layer thickness (mm)</th>
<th>After 10 pulses</th>
<th>After 340 pulses</th>
<th>After 720 pulses</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC (MPa)</td>
<td>50</td>
<td>3200</td>
<td>3900</td>
<td>3600</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>100</td>
<td>45</td>
<td>44</td>
<td>45</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>200</td>
<td>45</td>
<td>44</td>
<td>45</td>
</tr>
<tr>
<td>Soil (MPa)</td>
<td>500</td>
<td>45</td>
<td>44</td>
<td>45</td>
</tr>
<tr>
<td>Other (MPa)</td>
<td>∞</td>
<td>68947.6</td>
<td>68947.6</td>
<td>68947.6</td>
</tr>
</tbody>
</table>

*Table 54 Overview of the stiffness moduli of experiment 6*

*Figure 76: Calculated stiffness modulus of the asphalt concrete plate used in experiment 6*
A lower asphalt stiffness modulus was also found for the calculations with WESLEA. However, a constant stiffness modulus of the asphalt concrete could not be determined. It is observed that the stiffness modulus is different for each load magnitude, even for the same asphalt concrete plate on the same Baskarp sand bed. The lower stiffness modulus in the simulations of the medium scale experiments can be explained by:

- KOAC-NPC has performed 3 point bending tests for larger strains (50 \(\mu m/m\), 150 \(\mu m/m\), 300 \(\mu m/m\), and 500 \(\mu m/m\)) at 10°C to get to know if a higher strain results in a lower stiffness modulus for the asphalt concrete used in experiment 5. The results are presented in table 55.

<table>
<thead>
<tr>
<th>Number of tests</th>
<th>4</th>
<th>4</th>
<th>3</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain ((\mu m/m))</td>
<td>50</td>
<td>150</td>
<td>300</td>
<td>500</td>
</tr>
<tr>
<td>f (Hz)</td>
<td>(S_{mix}) (Mpa)</td>
<td>(S_{mix}) (Mpa)</td>
<td>(S_{mix}) (Mpa)</td>
<td>(S_{mix}) (Mpa)</td>
</tr>
<tr>
<td>0.1</td>
<td>3554</td>
<td>3393</td>
<td>2912</td>
<td>2713</td>
</tr>
<tr>
<td>0.2</td>
<td>4191</td>
<td>3944</td>
<td>3357</td>
<td>3062</td>
</tr>
<tr>
<td>0.5</td>
<td>5084</td>
<td>4794</td>
<td>4102</td>
<td>3716</td>
</tr>
<tr>
<td>1.0</td>
<td>5820</td>
<td>5488</td>
<td>4724</td>
<td>4277</td>
</tr>
<tr>
<td>2.0</td>
<td>6554</td>
<td>6225</td>
<td>5400</td>
<td>4884</td>
</tr>
<tr>
<td>5.0</td>
<td>7554</td>
<td>7242</td>
<td>6335</td>
<td>5698</td>
</tr>
<tr>
<td>8.0</td>
<td>8093</td>
<td>7787</td>
<td>6813</td>
<td>6026</td>
</tr>
<tr>
<td>10.0</td>
<td>8358</td>
<td>8052</td>
<td>7023</td>
<td>6073</td>
</tr>
<tr>
<td>20.0</td>
<td>9146</td>
<td>7782</td>
<td>6987</td>
<td>5957</td>
</tr>
<tr>
<td>30.0</td>
<td>9606</td>
<td>8735</td>
<td>7912</td>
<td>3911</td>
</tr>
</tbody>
</table>

*Table 55 Mean stiffness moduli at 10°C for different strains [KOAC-NPC, 2013]*

Higher strain levels result in a lower asphalt concrete stiffness modulus which has to be used in simulations [KOAC-NPC, 2013]. An increase of the load results in higher strains and thus a lower asphalt concrete stiffness modulus.

- The asphalt concrete plate thickness is not constant. Deltares has measured the thickness of the asphalt concrete plate, which shows that the 50 mm thick asphalt concrete plate can be up to 5 mm thicker or thinner. This can affect the elastic deformation profile as observed in experiment 6, where transducer V4 measures larger deflections than expected (and measured by transducer V5). Since the deflection profile is used for back-calculations, it also affects the stiffness modulus of the asphalt concrete.

- The accuracy of the elastic deformation measurements is also important to know how certain the shape of the elastic deformation profile is. This can also give uncertainty for the determined stiffness moduli of the asphalt concrete and sand bed.

From the back-calculations with WESLEA the stiffness modulus was found to be roughly between 40 MPa and 60 MPa.
4.4.6 Reflection on the WESLEA calculations/fitting

For the back-calculation with the WESLEA software the focus was on fitting the profile in order to get accurate values for the maximum strains rather than attaining a reasonable stiffness modulus for asphalt concrete. The following findings are given in doing this:

- For all 4 back-calculated experiments it can be observed that in general the strains decrease after repeated loading of the same magnitude, even for most of the loading stages of experiments 4 and 5.
- The area under the loading head at the underside of the asphalt layer is where the calculated strains are highest. This is also where cracks occurred during experiment 5 and 6.
- A constant stiffness modulus of the asphalt concrete could not be determined. It is observed that the stiffness modulus is different for each load magnitude, even for the same asphalt concrete plate on the same Baskarp sand bed. The stiffness modulus of the asphalt concrete plate depends on the strain level and the loading speed [KOAC-NPC, 2013]. The uncertainty of the asphalt concrete stiffness modulus also depends on the varying thickness of the asphalt concrete plate and the accuracy of the measured elastic deformations.
- The asphalt concrete stiffness also increases after a longer time of loading for experiments 3 and 4, but for experiments 5 and 6, where cracks were observed after the experiments, also a decrease of the asphalt stiffness moduli is observed.
- The stiffness modulus of the Baskarp sand bed used for the medium scale experiments roughly varies between 40 to 60 MPa depending on the loading conditions.
- It is observed that during the experiments the maximum (tensile) strains tends to decrease (as the Miner sum becomes larger than 1). This is likely if damage occurs to the asphalt plate as the plate cannot handle too large (tensile) strains. The (Baskarp) sand will have to have to resist more of the loading, which explains the displacement profile to get flatter in time. This can be observed from the displacement profiles of experiment 3 after 10 to 20 hours of loading.
- For experiment 3 after 10 and 51 hours of loading the layer of sand underneath the asphalt concrete slab tends to have a lower layer modulus than the sand layer(-s) below this layer. This may indicate that a small pit (or space) is formed in the sand right underneath the asphalt concrete plate. This can be the reason why the sand layer just underneath the asphalt concrete plate needs to be modeled with a lower layer modulus than the rest of the sand bed (as a combined stiffness modulus of sand and space/air).
- The maximum elastic deformations measured during experiment 4 are difficult to fit even after correction of rubber, because the measurements with V3 are relatively (very) deep. Reasonable fits are obtained with a SCI larger than 10 %. Assuming there
was a problem with transducer V5, also fits have been made excluding the measurements with transducer V5 for which it is observed that a better fit can be obtained. Since it is the second closest transducer to the center, it is very unfortunate not to take it into account for back-calculation. Excluding V5 obviously makes the fitting process less complicated and possibly less accurate.

- The nonlinear behavior of the sand (compaction of the sand) and asphalt concrete (viso-elastic behavior) is largest in the area underneath the loading head. Further from this area it is expected that the asphalt and the sand have a more linear elastic behavior. This can not be taken into account with the WESLEA software.

- For experiment 6 no failure is expected using the linear fatigue curves, but Deltares and KOAC-NPC used nonlinear fatigue curves for which the outcome predicted failure (in both cases the maximum strains were of similar magnitude of 456 $\mu m/m$ in WESLEA to 586 $\mu m/m$ with ABAQUS). After this experiment cracks were visible on the underside of the asphalt concrete plate. The use of nonlinear fatigue curves is advisable.

- Other than the high strain levels resulting in low stiffness moduli for the asphalt concrete, a lower stiffness modulus of the asphalt concrete can also be caused by a combination of a too small thickness of the asphalt slab used in the medium scale tests and the relative high percentage of bitumen in the asphalt mix in general. The bitumen which exhibits visco-elastic behavior together with the small slab thickness can allow for higher strains during the medium scale test to occur. When back-calculations are done, a low layer modulus will be found for the asphalt concrete, because the stiffness modulus of the bitumen in the asphalt mix will play a large role in holding the asphalt mix together under such loads. For this reason also a failure mechanism caused by the failure due to exceeding the shear strength of the asphalt concrete plate may occur.
Chapter 5 Calculations with ABAQUS software

A more detailed insight can be achieved with a finite element analysis software called ABAQUS. This software allows a more complex geometry to be defined and a variety of material models and input parameters can be applied. In this chapter an analysis is done with this software and it is also reflected on how the model in ABAQUS can be modified in order to simulate the medium scale tests in a better way.

5.1 Input parameters

In order to perform finite element calculations with the ABAQUS software a model of the test set up has been built [Davidse M.P., 2012]. The walls of the cylinder and the loading head are made infinitely stiff. The following geometry is created, which is roughly the basis of the model (see Figure 77).

![Model of the test set up in ABAQUS](image)

Figure 77: Model of the test set up in ABAQUS [Davidse M.P., 2012]

Different material models and parameter values can be assigned to the different parts of the geometry, i.e. the asphalt plate (green part in Figure 77) and the Baskarp sand underneath the asphalt (red part in Figure 77). The parameters, especially the stiffness parameters, are adjusted for fitting the output of the calculations to the measurement results of the medium scale tests. The ABAQUS software enables to investigate a variety of parameters which can influence the output. This makes the model more complex in order to gain more accuracy if applied correctly. The following paragraphs give an overview of the material models used with the assumed parameter values.
5.1.1 Asphalt
For the asphalt plate 2 material models are considered:
1. The linear elastic material model.
   This is one of the simplest material models which can be used to model the asphalt concrete plate. Nonlinear effects caused by the visco-elastic behavior of asphalt are excluded in this type of material model.
2. The visco-elastic material model.
The visco-elastic material model is used for the asphalt concrete plate, because the bitumen in asphalt makes the behavior of asphalt also visco-elastic. If this model is used correctly more insight could be gained about nonlinear behavior which could depend on time (more nonlinear effects are expected after longer time of loading of the asphalt concrete).

For the linear elastic model the parameters to be taken into account are:
- The stiffness modulus. The stiffness modulus has been determined with laboratory tests [KOAC-NPC, 2012] (see APPENDIX A). When fitting the output to the results, these values have proven to be too large as input for the models. This is also the case for the output of the calculations performed with WESLEA software.
- The density of the asphalt concrete plate. This has also been determined with laboratory tests and is different for the new and the old asphalt concrete (see APPENDIX A).
- Poisson’s ratio is taken as 0.35. This is a reasonable value for asphalt concrete.

The visco-elastic material model needs more input, because more characteristics of the material behavior will be simulated in the model. The same 3 input parameters as for the linear elastic material model are used. The extra input parameters needed are the so called Prony series. These are determined on the basis of the stiffness and the phase angle for each frequency which were determined during the stiffness tests [KOAC-NPC, 2013].

5.1.2 Baskarp sand
For the Baskarp sand also 2 material models are considered:
1. Linear elastic material model.
   This is one of the simplest material models which can be used to model Baskarp sand. Nonlinear behavior caused by compaction of the sand underneath the asphalt concrete plate is not taken into consideration using this model. In time the sand underneath the asphalt concrete plate subjected to loading is expected to react stiffer as it is being compacted, especially for the sand in the area directly underneath the loading head.
2. The modified Drucker-Prager material model (See APPENDIX F).
   This model can take the effect of the stiffening of the sand underneath the asphalt concrete plate into account by introducing more input parameters. This material
model can take plastic deformations into account, which is very useful as permanent deformations have been observed on the sand bed during and after the experiments when removing the asphalt plate.

For the linear elastic model the parameters to be taken into account are the same types as for the asphalt concrete plate, but with different values. The parameters are described for modeling the Baskarp sand:

1. The stiffness modulus. The stiffness modulus is found iteratively. The stiffness moduli found with the back-calculations with WESLEA can give good first estimates. While fitting the output to the results of the medium scale tests, this value may change.
2. The density of Baskarp sand. This value of the Baskarp sand has been checked on the basis of core sampling after the medium scale tests.
3. Poisson’s ratio is taken to be 0.25. This is an acceptable value for this type of sand.

For the modified Drucker-Prager material model the sand will behave elastically up to some state of (shear) stress at which slip or yielding occurs. This model will enable the effect of compaction of the sand, but requires more input parameters. The values for the parameters used in the model are introduced.

From triaxial tests [Davidse M.P., 2013] executed on Baskarp sand the following data is obtained:

- Angle of internal friction (ϕ) = 37°
- Cohesion = 1*10^-5 MPa
- Volumetric mass = 2000 kg/m³

The angle of internal friction for the Drucker-Prager material model used in ABAQUS is calculated with the following formula [Davidse M.P., 2013]:

\[
\tan\beta = \frac{6 \cdot \sin \phi}{3 - \sin \phi} \Leftrightarrow \beta_{dp} = 56.4°
\]

From the oedometer test (consolidation test) performed by Deltares, the following parameters are known about Baskarp sand [Davidse M.P., 2013]:

- Void ratio = 0.657. This is the value corresponding to lowest series and is used to determine the elasticity and hardening cap.
- Compression index (\(C_c\)) = 0.001657 (modified for ABAQUS input)
- Expansion index (\(C_s\)) = 0.0137 (modified for ABAQUS input)
The remaining input values for the Drucker-Prager material model were chosen by Deltares:

- Cap eccentricity (R) = 0.5
- Initial Yield Surf Pos (ε_{pl,0}) = 0
- Transition Surf Rad (α_T) = 0.05
- K = 0.8 (material property that determines the yielding)

In case of opting for the porous elastic material model more properties for input values in the ABAQUS model are necessary. These are determined [Davidse M.P., 2013] and given:

- Compression modulus: \( \kappa = 2 \cdot \frac{C_{s,e}}{2.3} = 0.002348 \) (using the unmodified \( C_{s,e} = 0.0027 \))
- A tensile limit (p_{t,e}) = 0.0001 MPa. This is chosen as low as possible as sand is assumed to have almost no tensile strength.
5.2 Results and analysis

The first calculations with this software are done with linear elastic material models for both the asphalt and the sand. The deflection profiles of experiment 5 are considered here. These first calculations can be compared to the back-calculations treated in section 4.3.3. That is why the same stiffness modulus found there is used for the asphalt concrete and an average of the stiffness modulus is used to model the sand.

Figure 78 shows the fit for the last loading of 2.4 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 560 MPa and for the sand 53 MPa.

**Figure 78: Measured elastic deformations vs. calculated with ABAQUS for experiment 5 for 2.4 kN**

Figure 79 shows the fit for the first loading of 5.7 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 800 MPa and for the sand 36 MPa.

**Figure 79: Measured elastic deformations vs. calculated with ABAQUS for experiment 5 for 5.7 kN**
Figure 80 shows the fit for the first loading of 5.9 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 1010 MPa and for the sand 46 MPa.

Figure 80: Measured elastic deformations vs. calculated with ABAQUS for experiment 5 for 5.9 kN

Figure 81 shows the fit for the first loading of 8.5 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 1010 MPa and for the sand 48 MPa.

Figure 81: Measured elastic deformations vs. calculated with ABAQUS for experiment 5 for 8.5 kN
Figure 82 shows the fit for the first loadings of 8.9 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 960 MPa and for the sand 51 MPa.

A reasonable fit is found for all of the loads, except for the loadings of 8.5 and 8.7 kN (especially fitting the measured elastic deformation of transducer V3). A higher asphalt stiffness modulus can make the fit more acceptable. Therefore a better fit is made with a higher asphalt concrete stiffness for these 2 loading stages.

Figure 83 shows the fit for the first loading of 8.5 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 1300 MPa and for the sand 48 MPa.
Figure 84 shows the fit for the first loadings of 8.9 kN during experiment 5. The stiffness modulus taken for the asphalt concrete is 1300 MPa and for the sand 51 MPa.

**Figure 84: Measured elastic deformations vs. calculated with ABAQUS for experiment 5 for 8.7 kN**

The elastic deformation profile is flattened in the area underneath the loadinghead (partly caused by the uniform loading).

The calculations with the WESLEA model yields layer stiffness moduli that are of similar magnitude compared to the calculations with the ABAQUS model where the sand is modelled as one layer. With this result further calculations can be done with the ABAQUS software using the other available material models.
The second calculations with this software are done with a linear elastic material model for the asphalt concrete plate and the Drucker-Prager material model for the Baskarp sand. The deflection profiles of experiment 5 are considered here. The calculations only completed successful for the loadings of 2.4 kN. For the other loadings the calculations aborted, because the solution appeared to be diverging. The result of the loading of 2.4 kN is given in figure 85.

![Vertical elastic deformations: measured vs. calculated through fitting](image)

*Figure 85: Measured elastic deformations vs. calculated with ABAQUS DP, experiment 5, 2.4 kN*

Using the input parameters given in 5.1 the calculated elastic displacements in the area close to the loading head are larger compared to the calculation done with a linear elastic material model for the Baskarp sand. A fit is made using a larger asphalt concrete stiffness modulus of 1100 MPa and the same layer stiffness modulus for the sand of 53 MPa (see figure 86).
Vertical elastic deformations: measured vs. calculated through fitting

![Graph: Measured elastic deformations vs. calculated with ABAQUS DP, experiment 5, 2.4 kN]

*Figure 86: Measured elastic deformations vs. calculated with ABAQUS DP, experiment 5, 2.4 kN*
5.3 Evaluation

- Using the input values from the WESLEA calculations, fits are made using a linear elastic material model for both the asphalt concrete plate and the sand bed. In order to get a proper fit for the measured elastic deformation profiles of the first loading of 8.5 kN and the last loading of 8.7 kN a larger asphalt concrete stiffness modulus is used.

- For the second type of calculations only the calculations from the loading of 2.4 kN succeeded. The calculations with the other loadings (of larger magnitude) did not complete successfully and are therefore not treated in this report. From the loading of 2.4 kN is is observed that larger elastic deformations are found. This is why a larger asphalt concrete stiffness modulus can be applied.

- The rubber between the loading head and the asphalt plate also deforms during loading. Therefore the measurements with transducer V3 need to be corrected (see APPENDIX D). As seen in the results of the ABAQUS calculations, the deflection profile of the surface of the asphalt concrete underneath the loading head is not straight. This means that the sides of the rubber get squeezed more than at the center. The correction due to the rubber for the measurement with transducer V3 can be different. This can also explain why the measurement with transducer V3 is smaller than the measurement with transducer V5 for experiment 3 after 51 hours of loading.

- From the WESLEA calculations a better understanding of the behavior of the asphalt concrete plate has led to better insights. It has become obvious that the nonlinear behavior of the sand underneath the asphalt concrete plate plays a large role for the finite element calculations. The most nonlinear effects are detected in the area close to the loading head. This part is located in a radius of 100 to 200 mm from the load center and 300 mm deep (corresponding to the first 2 layers of sand in the WESLEA model). A proposal is presented in figure 87 for which it is chosen to model this part of the sand with the Drucker-Prager material model (orange part in Figure 87). The rest of the sand can be modeled by either the linear elastic material model or the Drucker-Prager material model. The asphalt concrete plate can be modeled with the visco-elastic material model. The intention of this proposed modified model is to improve the accuracy of the calculation and maybe even prevent unwanted errors during calculations.
Figure 87: Modified model
Chapter 6 Conclusions and recommendations

In this chapter the conclusions and recommendations regarding this study are presented.

6.1 Conclusions

The linear elastic calculations with the WESLEA software led to a better understanding of the behavior of the asphalt concrete plate on the Baskarp sand for both the old and new asphalt concrete. The following conclusions can be drawn regarding the WESLEA calculations:

- A constant stiffness modulus of the asphalt concrete could not be determined. It is observed that the stiffness modulus is different for each load magnitude, even for the same asphalt concrete plate on the same Baskarp sand bed. The stiffness modulus of the asphalt concrete plate depends on the strain level and the loading speed [KOAC-NPC, 2013]. The uncertainty of the asphalt concrete stiffness modulus also depends on the varying thickness of the asphalt concrete plate and the accuracy of the measured elastic deformations.

- From the back-calculations of the medium scale experiments with WESLEA typical stiffness moduli for the Baskarp sand bed roughly varying between 40 to 60 MPa are found, depending on the loading conditions. These values are also sufficient for the linear elastic calculations with ABAQUS model for experiment 5.

- It is observed that during the experiments the maximum (tensile) strains tends to decrease. This is likely if damage occurs to the asphalt plate as the plate cannot handle too large (tensile) strains. The (Baskarp) sand will have to have to resist more of the loading, which explains the displacement profile to get flatter in time. This is observed from the displacement profiles of experiment 3 after more than 10 hours of loading.

- The maximum elastic deformations measured during experiment 4 are difficult to fit even after correction of rubber, because the measurements with V3 are relatively (very) deep. Reasonable fits are obtained with a SCI larger than 10%. Assuming there was a problem with transducer V5, also fits have been made excluding the measurements with transducer V5 for which it is observed that a better fit can be obtained. Since it is the second closest transducer to the center, it is very unfortunate not to take it into account for back-calculation. Excluding V5 obviously makes the fitting process less complicated and possibly less accurate.

- For experiment 6 no failure is expected using the linear fatigue curves, but Deltares and KOAC-NPC used nonlinear fatigue curves for which the outcome predicted failure (in both cases the maximum strains were of similar magnitude of 456 µm/m in WESLEA to 586 µm/m with ABAQUS). After this experiment cracks were visible on the underside of the asphalt concrete plate. The use of nonlinear fatigue curves is advisable.

- The nonlinear behavior of the Baskarp sand bed (compaction) and asphalt concrete (viso-elastic behavior) is largest in the area underneath the loading head. Further
from this area it is expected that the asphalt and the sand have a more linear elastic behavior.

- The nonlinear behavior (compaction/stiffening) of the Baskarp sand underneath the asphalt concrete plate causes the sand bed to reshape which has its effect on the behavior of the asphalt concrete plate. The formation of a small pit (or space) underneath the asphalt concrete plate (in the area underneath the loading head) can lead to a lower stiffness modulus of the asphalt concrete and sand bed (as a combined stiffness modulus of sand and space/air). Possibly this occurred during experiment 3 after more than 10 hours of loading.

The following conclusions can be drawn regarding the ABAQUS calculations:

- No improvements of the ABAQUS model are made. But a proposal is given to improve the ABAQUS model in order to avoid unwanted errors when performing the calculations.
- Using the input values from the WESLEA calculations, fits are made using a linear elastic material model for both the asphalt concrete plate and the Baskarp sand bed. For the first loading of 8.5 kN and the last loading of 8.7 kN a larger asphalt concrete stiffness modulus is used. This shows that the linear elastic calculations performed with the WESLEA software and the ABAQUS software are both in agreement.
- For calculations with a Drucker-Prager material model for the sand bed only the calculations from the loading of 2.4 kN succeeded from which it follows that this material model provides a higher asphalt stiffness modulus that can be applied.

The following general conclusions can be drawn:

- Failure of an asphalt concrete plate does not necessarily have to result in cracks as other failure mechanisms such as failure by exceeding the shear strength and failure of the subsoil are also possible.
- In practice, the asphalt dike revetments are not experiencing the loading on one point of the revetment, but more spatially spread, while the medium scale experiments were executed with a loading on one point. The sand bed underneath an in situ asphalt revetment will compact over a larger area under the revetment, because the wave loads are spread over different points of impact on the revetment.
- Since most of the current asphalt concrete revetments in the Netherlands consist of aged asphalt concrete (about 90% is older than 20 years), the failure mechanism most likely to occur under wave impacts is failure due to fatigue. From the research done up till now, it has become clear that if the asphalt concrete revetments which are less affected by ageing are less vulnerable to failure due to fatigue under wave impacts.
### 6.2 Recommendations

In this paragraph the following recommendations regarding research on asphalt concrete revetments on a Baskarp sand bed are made:

- In order to get to know more about the interaction between an asphalt concrete plate and the Baskarp sand underneath, experiments can be conducted on an asphalt concrete plate and Baskarp sand bed separately:
  - An asphalt concrete plate with the same dimensions of the plate used in the medium scale experiments under the same loading conditions can be placed on a material with (almost) a linear elastic behavior of which the stiffness modulus is known.
  - A plate produced out of a homogenous material with a known linear elastic behavior and not as sensitive to temperature differences as asphalt concrete can be placed on the Baskarp sand bed using the same test set-up for the medium scale experiments under the same loading conditions.

- A re-check of the thickness of the asphalt plate can be considered. A too thin asphalt plate can lead to failure due to exceeding the shear strength of the asphalt concrete plate.

- Investigate the proposed improved model in ABAQUS to make calculations with it. The proposed improved model for ABAQUS is based on the findings of the back-calculations with WESLEA.

- The bottom (of concrete floor) under the sand has an effect on the elastic deformation profile. This needs to be taken into account for the design of the large scale experiments (see APPENDIX E).

- The accuracy of the correction for the rubber must be improved. The sides of the rubber get squeezed more than in the center, which means the exact deflection is determined in a more or less arbitrary manner.
Literature


Websites:

http://en.wikipedia.org/wiki/Asphalt, accessed on 24-4-2013


www.stowa.nl, accessed on 17-4-2013


http://www.tutorgigpedia.com/ed/Lake_Bermudez, accessed on 11-7-2013

http://origin-ars.els-cdn.com/content/image/1-s2.0-S0950061810006161-gr7.jpg, accessed on 5-7-2013
APPENDIX A: Asphalt properties

Input data new asphalt [KOAC-NPC, 2012]:

Tabel 2-2 Resultaten penetratie en verwekingspunt

<table>
<thead>
<tr>
<th>(Q) NEN-EN 1425</th>
<th>70/100 esso Antwerpen</th>
<th>Eenheid</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bepaling van de penetratie</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Penetratie in 0,1 mm</td>
<td>77</td>
<td></td>
</tr>
<tr>
<td>Temperatuur bij bep. penetratie</td>
<td>25.0 °C</td>
<td></td>
</tr>
<tr>
<td>(Q) NEN-EN 1427</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Bepaling van het verwekingspunt - Ring- en kogelmethode</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Verwekingspunt in °C</td>
<td>46.8 °C</td>
<td></td>
</tr>
</tbody>
</table>

(Q) NEN-EN 12591 Annex A
Specificaties voor penetratiebitumen, berekening van de penetratie index

Penetratie-index | -1.0 |

Tabel 2-3 Resultaat berekening op basis van nomogrammen

<table>
<thead>
<tr>
<th>Load Time (Hertz)</th>
<th>Bitumen Temp °C</th>
<th>Pen Value 0,1mm</th>
<th>Pen Temp °C</th>
<th>Softening Point °C</th>
<th>Pen Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>5</td>
<td>77</td>
<td>25</td>
<td>46,8</td>
<td>-1,00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bitumen Stiffnesss MPa</th>
<th>Vol % Bitumen %v/v</th>
<th>Vol % Aggregate %v/v</th>
<th>Mix Stiffness MPa</th>
<th>Fatigue Strain µm/m</th>
<th>Fatigue Life Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>207,0</td>
<td>14,30</td>
<td>85,40</td>
<td>15.200</td>
<td>300</td>
<td>81.500</td>
</tr>
</tbody>
</table>

Tabel 2-4 Resultaat berekening op basis van nomogrammen met proefopstelling

<table>
<thead>
<tr>
<th>Load Time (Hertz)</th>
<th>Bitumen Temp °C</th>
<th>Pen Value 0,1mm</th>
<th>Pen Temp °C</th>
<th>Softening Point °C</th>
<th>Pen Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>10</td>
<td>77</td>
<td>25</td>
<td>46,8</td>
<td>-1,00</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bitumen Stiffnesss MPa</th>
<th>Vol % Bitumen %v/v</th>
<th>Vol % Aggregate %v/v</th>
<th>Mix Stiffness MPa</th>
<th>Fatigue Strain µm/m</th>
<th>Fatigue Life Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>69,5</td>
<td>14,30</td>
<td>85,40</td>
<td>8.500</td>
<td>300</td>
<td>231.000</td>
</tr>
</tbody>
</table>
Table 2.7 Specifications of the mixture of the asphalt concrete plates in percentage of total mass.

<table>
<thead>
<tr>
<th></th>
<th>Experiment 1 [%]</th>
<th>Experiment 2, 3 and 4 [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>River sand</td>
<td>39.1</td>
<td>39.1</td>
</tr>
<tr>
<td>Wigro</td>
<td>7.1</td>
<td>7.1</td>
</tr>
<tr>
<td>Eigen Stof</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>morene 8/11 molen</td>
<td>9.4</td>
<td>16.7</td>
</tr>
<tr>
<td>morene 112-8</td>
<td></td>
<td>16.7</td>
</tr>
<tr>
<td>morene 4/8</td>
<td>12.8</td>
<td>12.8</td>
</tr>
<tr>
<td>Bestone 8/16</td>
<td>24.0</td>
<td></td>
</tr>
<tr>
<td>Bitumen 70/100</td>
<td>6.1</td>
<td>6.1</td>
</tr>
</tbody>
</table>

Table 2.2 Parameters for the FEM-calculation for experiments 1 and 2. (*) Estimated from mixture composition; (**) Estimated from nomograms.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter loading head</td>
<td>5 cm</td>
</tr>
<tr>
<td>Load</td>
<td>4 kN</td>
</tr>
<tr>
<td>Stiffness sand</td>
<td>70 Mpa</td>
</tr>
<tr>
<td>Stiffness asphalt (*)</td>
<td>2760 Mpa</td>
</tr>
<tr>
<td>Thickness sand</td>
<td>95 cm</td>
</tr>
<tr>
<td>Thickness asphalt</td>
<td>5 cm</td>
</tr>
<tr>
<td>Temperature</td>
<td>23 °C</td>
</tr>
<tr>
<td>Maximum tensile stress</td>
<td>1.89 MPa</td>
</tr>
<tr>
<td>Maximum tensile strain</td>
<td>456 µm/m</td>
</tr>
<tr>
<td>Predicted number of loadings, leading to failure (**)</td>
<td>10,000-40,000</td>
</tr>
<tr>
<td>Pulse</td>
<td>0.1 s</td>
</tr>
<tr>
<td>Rest period</td>
<td>9.9 s</td>
</tr>
</tbody>
</table>

Table 2.3 Parameters for the FEM-calculation for experiment 3. (*) From laboratory tests, see [6].

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter loading head</td>
<td>10 cm</td>
</tr>
<tr>
<td>Load</td>
<td>12 kN</td>
</tr>
<tr>
<td>Stiffness sand</td>
<td>80 to 200 Mpa</td>
</tr>
<tr>
<td>Stiffness asphalt (*)</td>
<td>10563 Mpa</td>
</tr>
<tr>
<td>Thickness sand</td>
<td>80 cm</td>
</tr>
<tr>
<td>Thickness asphalt</td>
<td>5 cm</td>
</tr>
<tr>
<td>Temperature</td>
<td>10 °C</td>
</tr>
<tr>
<td>Maximum tensile stress</td>
<td>4.2 to 5.1 MPa</td>
</tr>
<tr>
<td>Maximum tensile strain</td>
<td>265 to 318 µm/m</td>
</tr>
<tr>
<td>Predicted number of loadings, leading to failure</td>
<td>3,000-15,000</td>
</tr>
<tr>
<td>Pulse</td>
<td>0.1 s</td>
</tr>
<tr>
<td>Rest period</td>
<td>9.9 s</td>
</tr>
</tbody>
</table>
The results of the four-point bend tests:

<table>
<thead>
<tr>
<th>Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter loading head</td>
<td>10 cm</td>
</tr>
<tr>
<td>Load</td>
<td>12 kN</td>
</tr>
<tr>
<td>Stiffness sand</td>
<td>95 Mpa</td>
</tr>
<tr>
<td>Stiffness asphalt</td>
<td>10535 Mpa</td>
</tr>
<tr>
<td>Thickness sand</td>
<td>80 cm</td>
</tr>
<tr>
<td>Thickness asphalt</td>
<td>5 cm</td>
</tr>
<tr>
<td>Temperature</td>
<td>10 °C</td>
</tr>
<tr>
<td>Maximum tensile stress</td>
<td>4.88 Mpa</td>
</tr>
<tr>
<td>Maximum tensile strain</td>
<td>305 μm/m</td>
</tr>
<tr>
<td>Predicted number of loadings, leading to failure</td>
<td>360 (*)</td>
</tr>
<tr>
<td>Pulse</td>
<td>0.1 s</td>
</tr>
<tr>
<td>Rest period</td>
<td>9.9 s</td>
</tr>
</tbody>
</table>

*Table 2.4 Parameters for the FEM-calculations for experiment 4. Only loading step 3 (load = 12 kN) was considered. (*) This value is determined from the fatigue curve for this asphalt, see [7].

Tabel 2-6 Stijfheid (MPa) bij 5°C

<table>
<thead>
<tr>
<th>Gemiddelde</th>
<th>T [°C]</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>f [Hz]</td>
<td>ε [μm/m]</td>
<td>E* [MPa]</td>
</tr>
<tr>
<td>0,1</td>
<td>50</td>
<td>5871</td>
</tr>
<tr>
<td>0,2</td>
<td>50</td>
<td>7316</td>
</tr>
<tr>
<td>0,5</td>
<td>49</td>
<td>9364</td>
</tr>
<tr>
<td>1,0</td>
<td>50</td>
<td>10935</td>
</tr>
<tr>
<td>2,0</td>
<td>50</td>
<td>12515</td>
</tr>
<tr>
<td>5,0</td>
<td>50</td>
<td>14520</td>
</tr>
<tr>
<td>8,0</td>
<td>50</td>
<td>15533</td>
</tr>
<tr>
<td>10,0</td>
<td>50</td>
<td>16015</td>
</tr>
<tr>
<td>20,0</td>
<td>50</td>
<td>17408</td>
</tr>
<tr>
<td>30,0</td>
<td>50</td>
<td>18167</td>
</tr>
</tbody>
</table>

Tabel 2-7 Stijfheid (MPa) bij 20°C

<table>
<thead>
<tr>
<th>Gemiddelde</th>
<th>T [°C]</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>f [Hz]</td>
<td>ε [μm/m]</td>
<td>E* [MPa]</td>
</tr>
<tr>
<td>0,1</td>
<td>51</td>
<td>398</td>
</tr>
<tr>
<td>0,2</td>
<td>50</td>
<td>575</td>
</tr>
<tr>
<td>0,5</td>
<td>49</td>
<td>980</td>
</tr>
<tr>
<td>1,0</td>
<td>51</td>
<td>1473</td>
</tr>
<tr>
<td>2,0</td>
<td>49</td>
<td>2167</td>
</tr>
<tr>
<td>5,0</td>
<td>50</td>
<td>3451</td>
</tr>
<tr>
<td>8,0</td>
<td>51</td>
<td>4260</td>
</tr>
<tr>
<td>10,0</td>
<td>51</td>
<td>4679</td>
</tr>
<tr>
<td>20,0</td>
<td>50</td>
<td>6088</td>
</tr>
<tr>
<td>30,0</td>
<td>50</td>
<td>6966</td>
</tr>
</tbody>
</table>
**Input data old asphalt [KOAC-NPC, 2013]:**

### Tabel 5-6 Resultaten penetratie

<table>
<thead>
<tr>
<th>0.1 [mm]</th>
<th>20</th>
<th>21</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gemiddelde 0.1 [mm]</td>
<td>20.5</td>
<td></td>
</tr>
</tbody>
</table>

### Tabel 5-8 Resultaten penetratie-index

<table>
<thead>
<tr>
<th>-</th>
<th>-0.2</th>
<th>-0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gemiddelde [-]</td>
<td>-0.25</td>
<td></td>
</tr>
</tbody>
</table>

### Tabel 5-7 Resultaten verwekingspunt – Ring en kogelmethode

<table>
<thead>
<tr>
<th>[°C]</th>
<th>63.8</th>
<th>62.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gemiddelde [°C]</td>
<td>63.3</td>
<td></td>
</tr>
</tbody>
</table>

### Tabel 5-3 Resultaten dichtheid mengsel

<table>
<thead>
<tr>
<th>[kg/m³]</th>
<th>2.406</th>
<th>2.400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gemiddelde [kg/m³]</td>
<td>2.403</td>
<td></td>
</tr>
</tbody>
</table>

### K L 1 2 Eenheid

#### Gehalte aan bitumen van warm bereid asfalt, soxhletextractie (directe methode)

Bitumengehalte A 1 6.9 7.2 %(%/m)

percentage berekend naar “in”

#### Gehalte aan bitumen van warm bereid asfalt, soxhletextractie (directe methode)

Bitumengehalte A 1 6.4 6.7 %(%/m)

### (Q) RAW 2008 proef 6.5

#### Korrelverdeling (zeefproef) (droog-nat-droog)

<table>
<thead>
<tr>
<th>op C 22.4</th>
<th>A</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>op C 16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op C 11.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op C 8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op C 5.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op 2 mm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op 500 μm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op 180 μm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>op 63 μm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 63 μm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>zandgradering 2 mm - 500 μm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>zandgradering 500 μm - 180 μm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>zandgradering 180 μm - 63 μm</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### (Q) RAW 2008 proef 60.1

Dichtheid van het mineraal van asfalt

Dichtheid mengsel A 1 2406 2400 kg/m³

### Tabel 5-4 Dichtheid proefstuk

<table>
<thead>
<tr>
<th>Aantal</th>
<th>Proefstukken boven</th>
<th>Proefstukken onder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gemiddelde [kg/m³]</td>
<td>2.268</td>
<td>2.197</td>
</tr>
<tr>
<td>Standaard deviatie [kg/m³]</td>
<td>17</td>
<td>23</td>
</tr>
<tr>
<td>Maximum [kg/m³]</td>
<td>2.285</td>
<td>2.219</td>
</tr>
<tr>
<td>Minimum [kg/m³]</td>
<td>2.238</td>
<td>2.161</td>
</tr>
<tr>
<td>Parameters</td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------------------------------------</td>
<td>----------</td>
<td></td>
</tr>
<tr>
<td>Diameter loading head</td>
<td>10 cm</td>
<td></td>
</tr>
<tr>
<td>Load</td>
<td>8.5 kN</td>
<td></td>
</tr>
<tr>
<td>Stiffness sand (*)</td>
<td>60 Mpa</td>
<td></td>
</tr>
<tr>
<td>Stiffness asphalt</td>
<td>2500 Mpa</td>
<td></td>
</tr>
<tr>
<td>Thickness sand</td>
<td>80 cm</td>
<td></td>
</tr>
<tr>
<td>Thickness asphalt</td>
<td>5 cm</td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>10 °C</td>
<td></td>
</tr>
<tr>
<td>Maximum tensile stress</td>
<td>2.98 MPa</td>
<td></td>
</tr>
<tr>
<td>Maximum tensile strain</td>
<td>785 μm/m</td>
<td></td>
</tr>
<tr>
<td>Predicted number of loadings, leading to failure</td>
<td>354</td>
<td></td>
</tr>
<tr>
<td>Pulse duration</td>
<td>0.1 s</td>
<td></td>
</tr>
<tr>
<td>Rest period</td>
<td>9.9 s</td>
<td></td>
</tr>
</tbody>
</table>

*Table 2.4 Parameters for the FEM-calculations for experiment 5 on old asphalt. For the prediction an elastic-plastic material model was used for the sand.*

<table>
<thead>
<tr>
<th>Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter loading head</td>
<td>10 cm</td>
</tr>
<tr>
<td>Load</td>
<td>7.2 kN</td>
</tr>
<tr>
<td>Stiffness sand (*)</td>
<td>60 Mpa</td>
</tr>
<tr>
<td>Stiffness asphalt</td>
<td>2900 Mpa</td>
</tr>
<tr>
<td>Thickness sand</td>
<td>80 cm</td>
</tr>
<tr>
<td>Thickness asphalt</td>
<td>5 cm</td>
</tr>
<tr>
<td>Temperature</td>
<td>10 °C</td>
</tr>
<tr>
<td>Maximum tensile stress</td>
<td>2.58 MPa</td>
</tr>
<tr>
<td>Maximum tensile strain</td>
<td>586 μm/m</td>
</tr>
<tr>
<td>Predicted number of loadings, leading to failure</td>
<td>755</td>
</tr>
<tr>
<td>Pulse duration</td>
<td>0.1 s</td>
</tr>
<tr>
<td>Rest period</td>
<td>9.9 s</td>
</tr>
</tbody>
</table>

*Table 2.5 Parameters for the FEM-calculations for experiment 6 on old asphalt. For the prediction an elastic-plastic material model was used for the sand, see also [5]*
APPENDIX B: Displacement measurements

The measurement data in APPENDIX B is retrieved from the 3 reports by Luijendijk, M.S. and Wichman, B.G.H.M. about the findings of the medium scale tests.

Experiment 1 [Deltares, 2012]

The duration of experiment 1 was about 31 hours, so about 11200 loadings of 3.7 kN. The displacement measurements are measured for specific times and are presented in the graphs.

Measured permanent displacements are given in the figure below.

![Figure B1: Profile of permanent displacements-experiment 1 [Deltares, 2012]](image1)

The maximum elastic displacements due to pulsed loading at different time steps are given in the figure below.

![Figure B2: Elastic displacements for all transducers-experiment 1 [Deltares, 2012]](image2)
Experiment 2 [Deltas, 2012]

The duration of experiment 2 was about 54 hours, so about 19440 loadings of 4.1 kN. The permanent displacement measurements are measured for specific times and are presented in the figure below.

Figure B4: Profile of permanent displacements-experiment 2 [Deltas, 2012]

The maximum elastic displacements due to pulsed loading for specific times are given in the figures below.
Figure B5: Elastic displacements for all transducers-experiment 2 [Deltas, 2012]

Figure B6: Vertical elastic displacements-experiment 2 [Deltas, 2012]

**Experiment 3 [Deltas, 2012]**

The duration of experiment 3 was about 52 hours, so about 18720 loadings of 12.2 kN. The permanent displacement measurements are illustrated for specific times in the figure below.
The maximum elastic displacements due to pulsed loading are illustrated for specific times in the figures below.

Figure B7: Profile of permanent displacements-experiment 3 [Deltas, 2012]

Figure B8: Elastic displacements for all transducers-experiment 3 [Deltas, 2012]
Experiment 4 [Deltares, 2012]

The duration of experiment 4 was about $1.06 \times 10^5$ seconds, so about 10500 loadings for which different force levels were applied (from 2 to 20 kN). The permanent displacement measurements are presented in the figure below for specific times.
The maximum elastic displacements due to pulsed loading at every force level applied are given in the table and figure below.

<table>
<thead>
<tr>
<th>Load level (kN)</th>
<th>4</th>
<th>12</th>
<th>16</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement V3 (mm)</td>
<td>0.3</td>
<td>0.6</td>
<td>0.75</td>
<td>0.85</td>
</tr>
<tr>
<td>Displacement V5 (mm)</td>
<td>0.1</td>
<td>0.33</td>
<td>0.45</td>
<td>0.53</td>
</tr>
</tbody>
</table>

*Table B1: Maximum elastic displacements due to pulsed loading at every force level applied—experiment 4 [Deltares, 2012]*
Experiment 5 [Deltas, 2013]

The duration of experiment 5 was about 1.5 hours, so about 580 loadings. The permanent displacement measurements are given for specific times in the figures below. Different forces were applied: 2.1 kN, 5.5 kN and 8.5 kN.

![Profile of permanent displacements - experiment 5 [Deltas, 2013]](image)

**Figure B13: Profile of permanent displacements - experiment 5 [Deltas, 2013]**

The maximum elastic displacements due to pulsed loading at every force level applied are given in the table and figure below.

<table>
<thead>
<tr>
<th>Load level (kN)</th>
<th>2.1</th>
<th>5.9</th>
<th>8.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement V3 (mm)</td>
<td>0.29</td>
<td>0.46</td>
<td>0.61</td>
</tr>
<tr>
<td>Displacement V5 (mm)</td>
<td>0.10</td>
<td>0.27</td>
<td>0.41</td>
</tr>
</tbody>
</table>

**Table B2: Maximum elastic displacements due to pulsed loading at every force level applied - experiment 5 [Deltas, 2012]**
Experiment 6 [Deltas, 2013, B]

The duration of experiment 6 was about 2 hours, 726 loadings. The permanent displacement measurements are given for specific times in the figures below. Different forces were applied during the test phase and experiment itself.

Figure B15: Profile of permanent displacements-experiment 6 [Deltas, 2013]

The maximum elastic displacements due to pulsed loading at every force level applied are given in the figure below.
Figure B16: Vertical elastic displacements-experiment 6 [Deltares, 2013]
APPENDIX C: Example of fitting procedure (Experiment 5, 8.7 kN)

During each experiment the maximum elastic deformations were measured. The table below gives an overview of the measured values in experiment 5 that will be analyzed by means of the linear-elastic multilayer program WESLEA. These deformations were measured for the last loadings of 8.7 kN on a loading head with a diameter of 100mm.

<table>
<thead>
<tr>
<th>Distance from center (mm)</th>
<th>maximum elastic displacement (micrometer)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V3 0</td>
<td>610</td>
</tr>
<tr>
<td>V5 100</td>
<td>410</td>
</tr>
<tr>
<td>V2&amp;V4 200</td>
<td>190</td>
</tr>
<tr>
<td>V1 350</td>
<td>70</td>
</tr>
<tr>
<td>V6 418</td>
<td>50</td>
</tr>
</tbody>
</table>

*Table C1: Measured maximum elastic deformation during experiment 5*

An effective way of finding the proper fit and values is by fitting the measurements from transducers V3 and V2&V4 and transducers V3 and V5 separately. With the values found for the input values, estimations can be made to find the possible values. This is done and the results are given below.
Table 16 gives the results from fitting V3 and V2&V4.

<table>
<thead>
<tr>
<th>CALCULATED THROUGH FITTING</th>
<th>FITTING TO MEASURED DEFLECTIONS</th>
<th>V3-V2&amp;V4</th>
</tr>
</thead>
<tbody>
<tr>
<td>y: distance from center (mm)</td>
<td>z: vertical distance/depth (mm)</td>
<td>y: vertical elastic displacement (micrometer)</td>
</tr>
<tr>
<td>0</td>
<td>0,5</td>
<td>610,91</td>
</tr>
<tr>
<td>0</td>
<td>49,5</td>
<td>576,17</td>
</tr>
<tr>
<td>101</td>
<td>0,5</td>
<td>356,1</td>
</tr>
<tr>
<td>101</td>
<td>49,5</td>
<td>355,97</td>
</tr>
<tr>
<td>199,9</td>
<td>0,5</td>
<td>190,03</td>
</tr>
<tr>
<td>199,9</td>
<td>49,5</td>
<td>190,27</td>
</tr>
<tr>
<td>350</td>
<td>0,5</td>
<td>99,93</td>
</tr>
<tr>
<td>350</td>
<td>49,5</td>
<td>100,05</td>
</tr>
</tbody>
</table>

Table C2: Results from fitting V3 and V2&V4

The input values (Layer Modulus) found after the fitting V3 and V2&V4 are given in table 17.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>810</td>
</tr>
<tr>
<td>Soil</td>
<td>70</td>
</tr>
<tr>
<td>Soil</td>
<td>65</td>
</tr>
</tbody>
</table>

Table C3: Input for fitting V3 and V2&V4

A graph is produced to compare the calculated profile with the measured profile.

Figure C1: Measured values vs. calculated values from fitting V3 and V2&V4
A fit is also made for the measurements of transducers V3 and V5. Table 18 gives the results of this fit.

<table>
<thead>
<tr>
<th>CALCULATED THROUGH FITTING</th>
<th>FITTING TO MEASURED DEFLECTIONS</th>
<th>V3-V5</th>
</tr>
</thead>
<tbody>
<tr>
<td>y: distance from center (mm)</td>
<td>z: vertical distance/depth (mm)</td>
<td>vertical elastic displacement (micrometer)</td>
</tr>
<tr>
<td>0</td>
<td>0,5</td>
<td>607,24</td>
</tr>
<tr>
<td>0</td>
<td>49,5</td>
<td>588,08</td>
</tr>
<tr>
<td>101</td>
<td>0,5</td>
<td>414,2</td>
</tr>
<tr>
<td>101</td>
<td>49,5</td>
<td>414,27</td>
</tr>
<tr>
<td>199.9</td>
<td>0,5</td>
<td>249,91</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>250,09</td>
</tr>
<tr>
<td>350</td>
<td>0,5</td>
<td>135,15</td>
</tr>
<tr>
<td>350</td>
<td>49,5</td>
<td>135,24</td>
</tr>
</tbody>
</table>

*Table C4: Results from fitting V3 and V5*

The input values (Layer Modulus) found after the fitting V3 and V5 are given in table 19.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>1386</td>
</tr>
<tr>
<td>Soil</td>
<td>61</td>
</tr>
<tr>
<td>Soil</td>
<td>50</td>
</tr>
<tr>
<td>Soil</td>
<td>50</td>
</tr>
</tbody>
</table>

*Table C5: Input for fitting V3 and V5*

A graph is produced to compare the calculated profile with the measured profile.

*Figure C2: Measured values vs. calculated values from fitting V3 and V5*

With the obtained input values of the separate fitting a rough estimation can be made for the input values for the best fit. This is done by taking the average of the input values found from fitting the measurements of transducers V3 with V5 and V2&V4 separately.
The averaged input values (Layer Modulus) for the last loads of 8.7 kN of experiment 5 are given in table 20.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>1098</td>
</tr>
<tr>
<td>Soil</td>
<td>65.5</td>
</tr>
<tr>
<td>Soil</td>
<td>57.5</td>
</tr>
<tr>
<td>Soil</td>
<td>57.5</td>
</tr>
</tbody>
</table>

*Table C6: The average input values for experiment 5*

The results of calculations with WESLEA using these average input values are given in table 21.

<table>
<thead>
<tr>
<th>y: distance from center (mm)</th>
<th>z: vertical distance/depth (mm)</th>
<th>vertical elastic displacement (micrometer)</th>
<th>Normal MicroStrain (x-direction)</th>
<th>Normal MicroStrain (y-direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.5</td>
<td>598.23</td>
<td>1060.39</td>
<td>1060.39</td>
</tr>
<tr>
<td>0</td>
<td>49.5</td>
<td>573.43</td>
<td>-1176.24</td>
<td>-1176.24</td>
</tr>
<tr>
<td>101</td>
<td>0.5</td>
<td>381.46</td>
<td>477.5</td>
<td>-117.3</td>
</tr>
<tr>
<td>101</td>
<td>49.5</td>
<td>381.45</td>
<td>-478.81</td>
<td>189.93</td>
</tr>
<tr>
<td>199.9</td>
<td>0.5</td>
<td>217.59</td>
<td>143.09</td>
<td>-183.39</td>
</tr>
<tr>
<td>199.9</td>
<td>49.5</td>
<td>217.8</td>
<td>-126.36</td>
<td>194.99</td>
</tr>
<tr>
<td>350</td>
<td>0.5</td>
<td>115.94</td>
<td>31.97</td>
<td>-62.11</td>
</tr>
<tr>
<td>350</td>
<td>49.5</td>
<td>116.05</td>
<td>-21.54</td>
<td>59.87</td>
</tr>
</tbody>
</table>

*Table C7: Result for the average input values*

Compared to the measured values, this gives a reasonable fit for the maximum elastic deformations (see figure 46).
A final fit is made using these values as a first estimation. The results are given in the tables and figure below:

<table>
<thead>
<tr>
<th>CALCULATED THROUGH FITTING</th>
<th>FINAL FIT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>y</strong>: distance from center (mm)</td>
<td><strong>z</strong>: vertical distance/depth (mm)</td>
</tr>
<tr>
<td>0</td>
<td>0,5</td>
</tr>
<tr>
<td>0</td>
<td>49,5</td>
</tr>
<tr>
<td>101</td>
<td>0,5</td>
</tr>
<tr>
<td>101</td>
<td>49,5</td>
</tr>
<tr>
<td>199,9</td>
<td>0,5</td>
</tr>
<tr>
<td>199,9</td>
<td>49,5</td>
</tr>
<tr>
<td>350</td>
<td>0,5</td>
</tr>
<tr>
<td>350</td>
<td>49,5</td>
</tr>
</tbody>
</table>

**Table C8: Results from final fitting**

The input values (Layer Modulus) found after the final fitting are given in table 23.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>1360</td>
</tr>
<tr>
<td>Soil</td>
<td>34</td>
</tr>
<tr>
<td>Soil</td>
<td>65</td>
</tr>
</tbody>
</table>

**Table C9: Input for final fitting**

![Vertical elastic deformations: measured vs. calculated through fitting](image)

**Figure C4: Measured values vs. calculated values from final fitting**
APPENDIX D: Correction for transducer V3

A correction needs to be taken into account regarding the rubber underneath the loading head. Since transducer V3 is mounted on the loading head, the measurements of this transducer will include the elastic deformation of the rubber. In order to determine the corrections tests on the rubber have been performed for different load magnitudes (see figure D1)

![Displacement measurements of the rubber for 5, 10, 15 and 20 kN](image)

*Figure D1: Displacement measurements of the rubber for 5, 10, 15 and 20 kN [Deltas, 2013]*

If a linear elastic behavior is assumed for the rubber (constant stiffness modulus), the corrections for measurements with V3 can be found through linear interpolation for each load. The displacement for each load are given in figure D2 for which a linear trendline is determined.

![Trendline for the displacement measurements of the rubber](image)

*Figure D2: Trendline for the displacement measurements of the rubber*

\[
\begin{align*}
\text{Displacement of rubber} & \\
\text{Load (kN)} & \\
\text{Elastic displacement (mm)} & \\
0 & 0.05 \\
5 & 0.10 \\
10 & 0.15 \\
15 & 0.20 \\
20 & 0.25 \\
25 & \\
\text{y} & = 0.0081x + 0.0375 \\
\text{R}^2 & = 0.9918 \\
\end{align*}
\]
From the trendline found in figure D2 the correction for the loads applied during the medium scale tests are determined through linear interpolation. The corrections are given for every applied load for experiment 3, 4 and 5 in table D1 (the correction of the measurements in experiment 6 was already done by Deltares).

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Correction (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experiment 3</td>
<td>12.2</td>
</tr>
<tr>
<td>Experiment 4</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td>4.3</td>
</tr>
<tr>
<td></td>
<td>12.4</td>
</tr>
<tr>
<td></td>
<td>16.25</td>
</tr>
<tr>
<td></td>
<td>19.85</td>
</tr>
<tr>
<td>Experiment 5</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td>5.9</td>
</tr>
<tr>
<td></td>
<td>8.5</td>
</tr>
<tr>
<td></td>
<td>8.7</td>
</tr>
</tbody>
</table>

*Table D1: Correction for transducer V3*
APPENDIX E: Influence of a stiff bottom layer on the deflection profile

In this APPENDIX the influence of the stiff bottom layer for layer 5 in the back-calculations with WESLEA is treated. The measurements during experiment 3 at t= 10 min are used to make clear what the effect of the stiff bottom compared to no stiff bottom (continuous layer stiffness modulus) is on the elastic deflection profile. The input is given in table E1 and the results are illustrated in figure E1.

<table>
<thead>
<tr>
<th>Input</th>
<th>With stiff bottom</th>
<th>Without stiff bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1 AC (MPa)</td>
<td>1380</td>
<td>1380</td>
</tr>
<tr>
<td>Layer 2 Soil (MPa)</td>
<td>80</td>
<td>80</td>
</tr>
<tr>
<td>Layer 3 Soil (MPa)</td>
<td>61</td>
<td>61</td>
</tr>
<tr>
<td>Layer 4 Soil (MPa)</td>
<td>61</td>
<td>61</td>
</tr>
<tr>
<td>Layer 5 Other/soil (MPa)</td>
<td>68947.6</td>
<td>61</td>
</tr>
</tbody>
</table>

*Table E1: Input values*

*Figure E1: Calculated elastic deflection profile for experiment 3 at 10 hours with and without stiff bottom layer*

It is observed that if a stiff bottom (layer 5 with a high stiffness modulus) in the model for back-calculations with WESLEA is taken, the magnitude of the elastic deflections are smaller than in case no stiff bottom is applied. The smaller thickness of the sand bed underneath the asphalt concrete plate acts as a shorter spring resulting in a smaller elastic deflection using a linear elastic model.
APPENDIX F: Modified Drucker-Prager material model

In this APPENDIX the modified Drucker-Prager model is treated as done in report e130041601 [KOAC-NPC, 2012].

In the Drucker-Prager model a failure plane is defined and if exceeded (failure by shear forces) compaction of the material and volume dilatancy occur. This failure plane is given in figure F1.

The parameters in figure F1 are defined as follows:

- $t$ = deviatoric stress (shear stress)
- $p$ = equivalent compressive stress
- $d$ = cohesion
- $\beta$ = modified angle of friction
- $F_s$ = shear failure
- $F_t$ = failure by transition of shear and/or compaction
- $F_c$ = failure due to compaction
- $\alpha$ = parameter for the shape of the transition
- $R$ = parameter for the shape of the cap
- $P_a$ = parameter for volumetric inelastic strain caused by hardening or softening
- $P_b$ = hydrostatic yield stress