Stellingen

behorende bij het proefschrift

EPS as a Light-Weight Sub-base Material in Pavement Structures

van

Milan Duškov

Delft, 18 juni 1997
Polystyreen hardschuim (EPS) is het enige lichte ophoogmateriaal dat algemeen toepasbaar is in lichtgewicht wegunite. In bepaalde gevallen kan schuimbeton een alternatief zijn.

Indien de verticale stuiw in EPS ten gevolge van verkeersbelasting beperkt blijft tot 0,4% zal permanente vervorming en stijfheidsverlies van EPS blokken van geen betekenis zijn. Genoemde waarde kan als ontwerpcriterium worden gebruikt.

De elasticiteitsmodulus noch andere eigenschappen van EPS gaan achteruit ten gevolge van omgevingsinvloeden zoals vochttopname of vries-dooi cycli.

Zowel de dikte van de EPS laag als de toepassing van een zwaarder (en duurder) type EPS, met een enigszins hogere elasticiteitsmodulus, hebben nauwelijks invloed op het structureel gedrag van wegconstructies met een EPS laag.

Een direct op een EPS-laag aangebracht ongebonden funderingsmateriaal vertoont een lagere 'effectieve' stijfheid dan indien hetzelfde materiaal wordt aangebracht op een (cement gebonden) stijve laag welk ligt op het EPS-hardshuur. De toepassing van zo'n stijve laag bovenop de EPS blokken maakt lichtgewicht constructies geschikt voor zwaarbelaste verhardingen, zelfs voor autosnelwegen.

3-D eindige elementen analyses van verschillende wegconstructies geven aan dat de toepassing van wapening in de ongebonden fundering het constructieve gedrag van wegconstructies met uitsluitend ongebonden materialen boven de EPS laag op een kosteneffectieve manier kan verbeteren.

Gegeven de kracht van de huidige generatie P.C.'s is het niet reëel meer om eindige elementen berekeningen aan verhardingsconstructies als niet praktisch te kwalificeren.

Voor zelfvertrouwen is geen hoge achting van zichzelf nodig. Het is voldoende een niet te hoge achting van anderen te hebben.

Om te voorkomen dat sommige politieke discussies over milieuproblematiek gaan lijken op de eerstjde discussies over het aantal engelen dat tegelijk op de top van een naald kan dansen, is het nodig om discussieargumenten te kwantificeren.

Het belangrijkste verschil tussen de oorlog tegen alcohol, gevoerd tussen 1920 en 1933, en de nog gaande maar eveneens verloren oorlog tegen drugs is dat het eind van de laatstgenoemde oorlog nog niet bekend is.

De belastingtarieven zouden alleen een ondergrens moeten zijn: sociaal bewuste medeburgers moeten de mogelijkheid krijgen om een voorbeeldfunctie te vervullen als zij dit willen.

Het toekennen van het zelfbeschikkingsrecht aan een niet homogene regio zonder hetzelfde recht toe te kennen aan de subregio's om binnen de oorspronkelijke staat te blijven is een voldoende voorwaarde voor een burgeroorlog.

De bedoeling van onze verlopen dagen is om hun stempel op vandaag te drukken.
Expanded polystyrene (EPS) geofoam is the only material that can be applied in all conditions where a light-weight pavement structure is required. In some cases foamed concrete can be an alternative.

If the vertical compressive strain in the EPS blocks due to traffic loads is limited to 0.4%, permanent deformation and reduction of the stiffness of the EPS blocks will be insignificant. This value can be taken as a design criterion.

Neither the elastic modulus of EPS nor its other characteristics are affected by environmental influences like repeated wetting and freeze-thaw cycles.

The thickness of the EPS layer as well as application of a denser (and more costly) EPS type, with a somewhat higher elasticity modulus, in the sub-base have only a very limited influence on the overall behaviour of pavement structures with an EPS sub-base.

If unbound roadbase materials are laid directly on top of the EPS sub-base they have a lower 'effective' stiffness than in case they are placed on a (cement bound) stiff layer which is laid on top of the EPS sub-base. Through the implementation of such a stiff capping layer above the EPS sub-base, light-weight structures are suited for heavily loaded pavements, even for motorways.

As resulting deformations in 3-D pavement models indicate, application of reinforcement in granular base layers would improve in a cost effective way the overall performance of pavement structures with only an unbound base above the EPS sub-base.

Given the power of the present generation of P.C.'s, it is not realistic anymore to qualify finite element calculations on pavement structures as being not practical.

For selfconfidence a high opinion about ourselves is not necessary. It is enough not to have a too high opinion about others.

In order to avoid that some political discussions about environment are similar to those concerning the number of angels that can simultaneously dance on the top of a needle, it is necessary to quantify the arguments to be discussed.

The most important difference between the war against alcohol, waged between 1920 and 1933, and the still going but also lost war against drugs is that the end of the last one is not known yet.

The tax rates should only be an official bottom limit: fellow citizens with social consciousness should get the opportunity to fulfil an exemplary function if they wish to do so.

Recognition of the right of self-determination to an inhomogeneous region without recognition of the identical right of self-determination to a subregion to stay within the original state creates a sufficient condition for a civil war.

The point of our yesterdays is to leave their mark upon today.
EPS AS A LIGHT-WEIGHT SUB-BASE MATERIAL IN PAVEMENT STRUCTURES

PROEFSCHRIFT

ter verkrijging van de graad van doctor aan de Technische Universiteit Delft, op gezag van de Rector Magnificus, Prof.dr.ir. J. Blauwendraad, in het openbaar te verdedigen ten overstaan van een commissie door het College van Dekanen daartoe aangewezen, op woensdag 18 juni 1997 te 13.30 uur
door

Milan DUŠKOV

civiel ingenieur
geboren te Belgrado, Joegoslavië
Dit proefschrift is goedgekeurd door de promotoren:

Prof.dr.ir. A.A.A. Molenaar en
Prof.dr.ir. Y.M. de Haan

Samenstelling promotiecommissie:

Rector Magnificus, voorzitter
Prof.dr.ir. A.A.A. Molenaar, TU Delft (promotor)
Prof.dr.ir. Y.M. de Haan, em. hgl. (1995), TU Delft (promotor)
Prof.dr.ir. A. Posthuma de Boer, TU Delft
Prof.ir. F.M. Sanders, TU Delft
Prof.ir. W.A. Segeren, TU Delft/IHE Delft
Prof.ir. A.F. van Tol, TU Delft
Ir. T. van Dorp, Shell Chemicals, Londen

Ir. L.J.M. Houben heeft als begeleider in belangrijke mate aan het totstandkomen van het proefschrift bijgedragen.

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Copyright © 1997 by Milan Duškov
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To Rade Zuppicich (1932 †1996),
a memorable man
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To you Sonja I am thankful for your understanding and support all these years. And thank to my parents and sister for their encouragement during my education. Last but surely not lest, I want to express my gratitude to my laid uncle Rade and Conni for their patronage during my study in Delft.
Summary

MILAN DUŠKOV, EPS as a Light-Weight Sub-base Material in Pavement Structures (1997)

This thesis deals with the use of Expanded Polystyrene (EPS) Geofoam as a light-weight sub-base material in pavement structures. Through a substantial reduction of pavement's weight, EPS as a sub-base material offers a major new solution for reduction of the settlements of new road structures and roads to be widened in areas with soils of poor load-bearing capacity. The application of EPS however affects the performance of the overlaying structure. In comparison with other sub-base materials EPS has, besides an extremely low density and a low modulus of elasticity, a low water absorption and a low thermal conductivity. To investigate, on one hand, to which extent the EPS characteristics influence the overall pavement behaviour and, on the other, the long term durability of EPS in relation to varying environmental conditions, materials research on EPS, in-situ measurements and numerical analyses of the structural performance of pavements with an EPS sub-base have been carried out.

The extensive testing of the EPS material (Chapter 5) involved the characterization of the elastic and permanent deformation behaviour under both repetitive and static loadings, the water absorption of EPS, as well as the mechanical properties of EPS15 and EPS20 after water absorption and freeze-thaw cycles. The experimental results can be summarized as follows: • EPS, if not overloaded and with an undamaged cell structure, absorbs water very slowly and to a very limited extent (few percents by volume). • Under the dead weight of thick pavement layers, the creep of both EPS20 and EPS15 amounts to no more than a few tenths of a percent. Therefore, the effect of creep of the EPS sub-base layer is of minor practical importance. • The dynamic E modulus of EPS20 under representative loading conditions is equal to approximately 9 MPa. The dynamic E-modulus of EPS15 is about 5.5 MPa. • As long as the elastic deformation due to traffic loads is limited to 0.4%, then permanent deformation of the EPS blocks will be insignificant and has no influence on pavement performance. • The Poisson's ratio value of EPS20 of 0.10 is appropriate for design purposes. • Negative influence on the mechanical behaviour of EPS due to low temperatures, water absorption and exposure to freeze-thaw cycles, separately or combined, has not been observed.

EPS has successfully been used as sub-base material in both asphalt and concrete block pavements. However, the consultants and constructors should be aware of some specific features of the EPS sub-base. For instance the very low E-values that were back-calculated from deflection measurements (Chapters 6, 7 and 8) indicate that for unbound base layers lower modulus values should be used than normally adopted. This problem has not been received sufficient attention yet. Furthermore, open joints between the EPS blocks in a sub-base can create very serious consequences for the design life of
pavement structures and therefore have to be avoided by all means. The longitudinal joints between the EPS blocks should not be close to a wheel track.

Three types of numerical analyses have been done. First of all, one-dimensional calculations of the temperature distribution in an asphalt pavement with a sand or an EPS sub-base have been performed. Secondly, the stresses, strains and displacements in a number of asphalt pavements were calculated. In these calculations the stress-dependent resilient and permanent deformation behaviour of the materials in the unbound pavement layers was taken into account. Finally, the characteristic block structure of the EPS sub-base had been implemented in 3-D finite element pavement models.

The main conclusion concerning temperature distribution (Chapter 3) based on the Dutch climate conditions is that in asphalt pavement structures with an EPS sub-base the asphalt temperature is approximately the same as in the corresponding pavement structures with a sand sub-base.

Based on the results of the finite element analyses (Chapters 6 and 9) it can be stated that: • The thickness of the EPS sub-base has only a marginal influence on the pavement design life. The stress and strain values in the upper pavement layers are almost the same in pavement structures with different thicknesses of the EPS layer. • Unbound roadbase materials have a much lower 'effective' stiffness in pavement structures without a load-spreading layer above the EPS sub-base compared to structures where a stiff layer (e.g. cement treated layer) is placed between the unbound base and the EPS layer. This effect should be taken into account when designing pavements with an EPS sub-base. • Implementation of a concrete capping layer above the EPS sub-base neutralizes the (negative) influence of the block joints on the pavement behaviour and improves the over-all pavement performance significantly. • Application of EPS types heavier than EPS15 in the sub-base has no significant influence on the strain values at the bottom of the asphalt layer. Therefore the use of these heavier types is not recommended.

In conclusion: pavements with an EPS sub-base can provide excellent performance of roads on soils with poor bearing capacity, provided that the design guidelines given in Chapter 10 are carefully obeyed.
Samenvatting

Milan Duškov, EPS als licht ophoogmateriaal in wegconstructies (1997)

Dit proefschrift handelt over de toepassing van geëxpandeerd polystyreen (EPS) hardschuim als licht ophoogmateriaal in wegconstructies. Door een substantiële gewichtsreductie van wegconstructies biedt EPS hardschuim een principieel nieuwe oplossing voor de beperking van zettingen bij de aanleg of verbreding van wegen op samendrukbare ondergrond. De implementatie van een EPS-laag beïnvloedt echter het gedrag van de bovenliggende verhardingslagen. Behalve een zeer lage volumieke massa en elasticiteitsmodulus verschilt EPS van traditionele ophoogmaterialen door een beperkte wateropname en zeer goede thermische isolatie-eigenschappen. De invloed van EPS op het structurele gedrag van verhardingen en de duurzaamheid van het EPS-hardschuim op lange termijn onder variërend condities in wegconstructies zijn onderzocht door middel van materiaalonderzoek op EPS, in-situ metingen en numerieke analyses van verhardingen met een EPS laag.

De uitgebreide beproeving van EPS-hardschuim (hoofdstuk 5) was gericht op de bepaling van de elastische en permanente deformaties van EPS onder zowel cyclische als statische belasting, de bepaling van de maximale wateropname van EPS en vaststelling van de invloed van wateropname en blootstelling aan vries/dooi cycli op de mechanische eigenschappen van EPS15 en EPS20. De proefresultaten kunnen als volgt worden samengevat:

- EPS absorbeert water in zeer beperkte mate (een paar procenten), mits de celstructuur niet beschadigd is door overbelasting.
- Onder een statische belasting, overeenkomend met het gewicht van de bovenliggende verhardingslagen, bedraagt de kruiw van EPS20 en EPS15 enkele tienden van een procent. Derhalve is de verticale deformatie van de verhardingsconstructie door kruiw van de EPS laag klein en van weinig praktische betekenis.
- De dynamische elasticiteitsmodulus van EPS20 onder representatieve belastingcondities in een EPS laag bedraagt circa 9 MPa. De elasticiteitsmodulus van EPS15 is gelijk aan 5,5 MPa.
- Zo lang de elastische rek in de EPS blokken ten gevolge van cyclische (verkeers)belasting beperkt blijft tot hooguit 0,4% is de optredende permanente deformatie in de EPS laag verwaarloosbaar.
- Lage temperaturen, wateropname en blootstelling aan vries/dooi cycli, afzonderlijk of gecombineerd, hebben geen negatieve invloed op het mechanisch gedrag van EPS. Voor ontwerpdoeleinden kan voor EPS een dwarscontractie-coëfficiënt van 0,1 worden aangenomen.

EPS wordt al op grote schaal met succes toegepast in zowel elementen- als asfaltverhardingen. Niettemin dienenwegbouwkundige adviseurs en uitvoerders zich bewust te zijn van sommige specifieke karakteristieken van de EPS-lagen. Zo duiden de lage E-moduli, teruggerekend op basis van valgewicht-deflectiemetingen (hoofdstukken 6, 7 en 8), op een lagere effectieve stijfheid van de direct op een EPS-laag aangebrachte ongebonden materialen dan wat hiervoor normaliter wordt aangenomen. Dit probleem is nog steeds
enigszins onderbelicht. Verder kunnen open voegen tussen de EPS-blokkken serieuze gevolgen hebben voor de levensduur van de verharding en moeten derhalve voorkómen worden. De longitudinale voegen tussen de blokkken mogen nooit dicht bij een wielpoor liggen.

Er zijn drie verschillende numerieke analyses uitgevoerd. Ten eerste zijn eendimensionale berekeningen van het temperatuurverloop in asfaltverhardingen met een EPS laag of een zandbed uitgevoerd. Ten tweede zijn spannings-, rek- en deflectiewaarden in asfaltverhardingen uitgerekend met een axiaal-symmetrische eindige elementen analyse, waarbij het spanningsafhankelijke materiaalgedrag in de ongebonden funderingslagen is gesimuleerd. Ten slotte is de karakteristieke blokstructuur in de EPS laag geïmplementeerd in een driedimensionaal eindige elementen model van asfaltverhardingen.

Ten aanzien van het temperatuurverloop in de asfaltverhardingen (hoofdstuk 3) onder gemiddelde Nederlandse klimaatcondities is de hoofdconclusie dat de temperatuur van het asfalt in de wegconstructies met een EPS-laag nagenoeg hetzelfde is als in de overeenkomende constructies met een zandbed.

Gebaseerd op de resultaten van de eindige elementen analyses (hoofdstukken 6 en 9) wordt het volgende gesteld: • De dikte van de EPS laag heeft een marginale invloed op het structureel gedrag van de verharding; de spanningen en rekken in de bovenlagen zijn nagenoeg onafhankelijk van de dikte van de EPS laag. • De direct op de EPS blokkken aangelegde ongebonden funderingsmaterialen hebben een lagere effectieve stijfheid in vergelijking met toepassing in wegconstructies met een lastspredende cementgebonden (beton)laag tussen de ongebonden funderingslagen en de EPS laag. Met dit effect moet rekening gehouden worden bij het ontwerp van een wegconstructie met een EPS laag. • Toepassing van een cementgebonden laag bovenop EPS neutraliseert de (negatieve) invloed van de voegen tussen de EPS blokkten en draagt aldus in belangrijke mate bij tot de verbetering van het structurele verhardingsgedrag. • Een zwaarder type EPS in de verhardingsconstructie heeft geen significante invloed op de rekwaarden onderin de asfaltaag. Derhalve is de toepassing van een EPS types zwaarder dan EPS15 niet nodig.

Samenvattend wordt gesteld dat door implementatie van een EPS laag op Weinig draagkrachtige, samendrukbare ondergrond duurzame lichtgewicht wegverhardingen met uitstekende constructieve eigenschappen ontworpen en aangelegd kunnen worden mits de ontwerprichtlijnen uit hoofdstuk 10 in acht worden genomen.
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CHAPTER 1

INTRODUCTION

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1.1 MOTIVATION FOR THE THESIS

Large areas of the western and northern parts of the Netherlands consist of subsoil with poor to very poor civil engineering characteristics. Figure 1.1 [1] shows Dutch areas with peat as the subsoil. It is important to underline the fact that most of the cities, e.g. Rotterdam, Amsterdam and Utrecht, are located in or surrounded by zones with such weak subsoils.

![Figure 1.1](image.png)

*Figure 1.1*

Schematic review of the areas in the Netherlands where the subsoil consists of peat

The peat layers were formed during the Holocene period and because flooding occurred regularly these peat layers are mixed with layers of compressible clay and silt. The schematic geotechnical profile of the Netherlands is given in Figure 1.2 [2]. Typical for regions with such soft subsoils are the large settlements of all the structures except those founded on piles.
Figure 1.2 - Schematic geotechnical profile through the Netherlands
The readings obtained by a Dutch cone penetration test performed in Stolwijk near Gouda (some 25 km east of Rotterdam) show the presence of thick peat layers (about 10 m over a deep sand) (Figure 1.3 [3]). Any road, railroad or dike structure built on top of such a subsoil is subjected to settlements and, even more important, uneven settlements which can reduce their serviceability. Especially uneven settlements cause a substantial increase of the costs of maintenance.

Figure 1.3 - Cone penetrometer test result in the Gouda region

These weak subsoils are further characterized by a high water content and a groundwater level close to the surface. Building embankments on such soft saturated subsoils in the
traditional way is time consuming because of the extra time needed to reduce the excess pore water pressures. The low shear resistance of saturated soils with excessive pore water pressure can also cause stability problems during the construction.

To reduce the extent of the settlement problems during road constructions, four alternative techniques have been developed. The dead weight of the road structure could be reduced, the subsoil could be either improved or replaced and the road structure could be founded on piles if the subsoil layers of bad quality are too thick. Shortcomings of the three lastly mentioned methods are elaborated in subsection 2.2.2. The most effective technique is to reduce the overburden pressure due to the road in such a way that there is a balance in weight of the excavated soil with the weight of the structure to be built.

Extremely light-weight materials like Expanded PolyStyrene (EPS) geofoam and, to a certain measure, light-weight cementitious materials, have the potential of solving the problem described above. Worldwide about 2 million m$^3$ of EPS have already been applied [4] for light-weight building purposes, 400,000 m$^3$ of which in the Netherlands in the past 10 years [5].

However, certain important questions in connection with the application of such light-weight materials, especially of EPS, are still to be answered. One of them is to what extent the low stiffness of EPS is affecting the performance of the overlying structure. Another question is related to the long term durability of such materials in relation to varying climatic conditions (moisture and frost). Finally there is the question of how pavement structures, containing a considerable amount of expanded polystyrene foam, behave under ongoing vibrations generated by traffic.

The above mentioned considerations lead to the initiation, in 1988, of a research project related to the behaviour of pavement structures where EPS is used as a sub-base material. The project was financed by Shell Nederland Chemie B.V. and, at a later stage, also by the Road and Hydraulic Engineering Division of the Ministry of Transport, Public Works and Watermanagament and Stybenex B.V. The research was performed at the Departments of Road Engineering (Road and Railroad Research Laboratory) and Material Science of the Faculty of Civil Engineering of the Delft University of Technology. The project, and its results, are described in this thesis.

### 1.2 SCOPE AND OBJECTIVES

Nowadays there are numerous applications of EPS geofoam in civil engineering. Apart from being a light-weight material it can provide also functions such as: thermal insulation, compressible inclusion, vibration damping and fluid transmission [6]. These additional functions will not be treated in the scope of the thesis.

EPS geofoam has been adopted as an extremely light-weight filling material in Norway in the early 70's. In those days it was intended to protect the frost-susceptible subsoil against freezing under the severe Scandinavian climatic conditions. Nowadays,
Introduction

EPS is built-in in road embankments, railway embankments, bridge abutments, self-standing walls, widening vertical walls, widening embankments for steep slopes, protection of structures etc. The objectives of the research program in this dissertation are however limited to the use of the EPS geofoam as a light-weight sub-base material in pavement structures which is at the same time the main application of EPS in road construction.

As mentioned in the foregoing, EPS has a high potential in reducing the amount of settlements in pavement structures and in reducing maintenance activities and costs related to settlements. On the other hand the use of such a light-weight, low modulus material might induce problems with respect to the performance of the overlaying structure if special precautions are not taken.

One of these aspects is for instance that it could be difficult to compact granular layers placed over the EPS sub-base to a sufficient density. This could result in base courses with a too low stiffness, which again could result in too high strains and early cracking in the asphalt layers if these layers are used in the same thickness as in the traditional pavements.

Another aspect is the temperature distribution in asphalt pavement structures. As an extremely good insulator, EPS influences the heat flow in a pavement structure in general, and in the sub-base in particular. The question is to what extent presence of the EPS sub-base affects the temperatures in the upper layers. Somewhat different temperatures in the asphalt layers would strongly influence the asphalt mix properties.

Furthermore, despite of the extensive prior use of EPS, insufficient information is available on the characteristics of the material itself, and especially on its long term performance under sub-base conditions. For instance, in a pavement structure EPS might be exposed to water for long periods of time. The question is whether or not water will penetrate the material as to increase the weight and to what extent the elastic and permanent deformation are influenced by e.g. moisture, frost, temperature, loading time and load repetitions.

Based on this problem description the following five objectives were defined for this research project.

a. Comparison of EPS with other light-weight sub-base materials, particularly foamed concrete.

b. Analysis of EPS20 and EPS15 with respect to elastic deformation under load, moisture sensitivity, frost sensitivity and permanent behaviour.

c. Characterization of pavement structures in which an EPS sub-base is built-in.

d. Analysis of in-situ pavements with respect to asphalt strains, elastic deflections and permanent deformations.

e. Definition of design guidelines for pavements with an EPS sub-base.
The above mentioned objectives were reached by the following research:

Re a. Inventory of light-weight sub-base materials was done (where granular materials, light-weight concrete types and plastic foams were distinguished) and their applicability was commented (Chapter 2). As a suitable sub-base material widely used in engineering practice, foamed concrete was considered in particular. Its properties were discussed as well as the results of the performed material testing (Chapter 4).

Re b. Extensive testing of EPS20 and EPS15 specimens has been performed which involved the characterization of the elastic and long term permanent deformation behaviour under repetitive and static loadings, the water absorption of EPS, the mechanical properties of EPS after water absorption and cyclic freeze/thaw tests (Chapter 5). Furthermore moduli assessments using ultra-sonic techniques were made to determine to what extent such a technique could be used for quality control purposes.

Re c. Extensive analyses have been made by means of the finite element programs DIANA and CAPA 3-D into the stresses and strains in flexible pavements having an EPS sub-base. In the DIANA analyses (Chapter 6) emphasis has been placed on the stress-dependency of unbound granular materials like sand, gravel etc. 3-D analyses (Chapter 9) were carried out to define the effects of typical EPS block patterns, the use of different EPS types and application of a concrete capping layer above the EPS sub-base on the pavement behaviour. Furthermore, because of the very low thermal conductivity of EPS the temperature distribution in pavement structures with an EPS sub-base was analyzed by means of the finite element program WEGTEM (Chapter 3).

Re d. Since the German Federal Highway Research Institute (BASf) was analyzing test pavements containing EPS built inside one of their laboratories, testing and analyzing of those different pavement types was done in close cooperation. Furthermore, an extensive deflection testing program has been performed on existing Dutch concrete block pavements (Chapter 7) and asphalt pavements (Chapter 8) with EPS sub-bases.

Re e. Based on the results of the research mentioned under a., b., c. and d. guidelines have been developed for the design of pavements using EPS as a sub-base material (Chapter 10).

It was decided to do the laboratory tests only on EPS20 and EPS15 (density 20 and 15 kg/m³). The sub-base of the analyzed concrete block pavement structures at Stolwijk consists partly of EPS20 and EPS15 while EPS25 and EPS30 are used as sub-base materials in the analyzed flexible pavement structure in Rotterdam.
1.3 OUTLINE OF THE THESIS

This thesis is organized as follows:

The light-weight foundation and fill materials applicable in a sub-base are reviewed and discussed in Chapter 2. Three groups of materials can be distinguished: granular materials, light-weight concrete types and plastic foams. A number of concluding remarks with respect to the applicability of the alternative light-weight sub-base materials are made at the end.

Chapter 3 contains a description of representative sub-base conditions regarding the temperatures, stress values and moisture contents. With respect to traditional sub-base materials (such as sand) those conditions differ in the case of EPS because of its much lower elasticity modulus as well as low thermal conductivity. The temperature distribution in pavement structures with an EPS sub-base was analyzed by means of the finite element program WEGTEM.

In Chapter 4 foamed concrete is discussed being an alternative light-weight sub-base material. The chapter contains information of the basic ingredients, the production process, material properties from the literature and results of performed material testing.

Chapter 5 reports the results of materials research on EPS15 and EPS20 geofoams. Firstly, details are given about the production process of EPS20 and EPS15 specimens on which the experiments were carried out. Secondly, water absorption of EPS20, as measured by means of immersion and exposing to freeze-thaw cycles, is described. Subsequently, the influence on the mechanical behaviour of EPS15 and EPS20 of water absorption and treatment under extreme temperatures is determined by means of compression tests. Furthermore, the creep in the EPS sub-base, investigated under representative stresses, is given. In the next section uniaxial cyclic loading tests are discussed. Finally, determination of the dynamic moduli of elasticity of EPS20 by electro-dynamic methods is described, and the overall conclusions pertaining to the materials research on EPS15 and EPS20 are drawn.

Chapter 6 gives details about six full-scale pavement test sections which were constructed at the German BASt. The author contributed to the measuring program by Falling Weight Deflectometer (FWD) tests on the test pavements, by triaxial loading tests on the granular base material and by extensive numerical analyses. The used finite element program DIANA contains a non-linear elastic material model suitable for simulation of the stress-dependent behaviour of unbound gravel materials. In this way the influence of a load-distributing intermediate layer on top of the EPS sub-base on the unbound roadbase material was analyzed.

In Chapter 7 the results of FWD measurements and visual inspection on concrete block pavement structures are presented. Graphical presentations of the FWD deflections as
a function of time give insight into the structural pavement behaviour in time and the influence of traffic. The FWD results were used for a back-calculation analysis which resulted in the moduli of elasticity of the pavement layers. Finally, general conclusions are drawn, which refer to the general condition of the block pavement structures, differences between the sections and remarks on the E-values found for the unbound materials and for EPS.

Chapter 8 deals with FWD measurements and asphalt strain measurements that were performed on the Matlingeweg in Rotterdam. This road was of big interest to investigate because of its 1.0 m thick EPS sub-base and heavy traffic loading. Four strain transducers were built-in at the bottom of the asphalt layer. Information about the development of the horizontal asphalt strain in a real pavement structure permits the analysis of its structural behaviour. Interpretation of the results was complicated because of the different temperatures during the measurements. The measured strain values had to be transformed to a reference temperature before comparison. At the end, the trend of the asphalt strain values (at the reference temperature of 20°C) is presented as a function of the pavement age.

Chapter 9 discusses three-dimensional (3-D) finite element analyses of pavement structures with an EPS sub-base layer. The third dimension is of importance since it allows modelling of the block pattern in the EPS sub-base. Three separate 3-D analyses were carried out. Firstly, a 3-D pavement structure model was developed with a single vertical interface layer modelled next to the wheel load. Secondly, a model for a polder road was developed to analyze the effects of: a) different block patterns, b) various EPS types in the sub-base and c) a concrete capping layer on the stress and strain values in the pavement layers. Finally, using experiences from the previous analyses, a model for a motorway pavement structure was designed to investigate the consequences of implementation of EPS (instead of sand) on the performance of (Dutch) motorway pavement structures. The analyses were realized by means of the three-dimensional version of the program CAPA.

The guidelines and recommendations for the design of pavements with an EPS sub-base, based on the considerations given in the previous chapters, are given in Chapter 10.

Finally, the main conclusions of this thesis, drawn from our research results and analyses, are summarized in Chapter 11.

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CHAPTER 2

LIGHT-WEIGHT SUB-BASE MATERIALS

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2.1 INTRODUCTION

As discussed in Chapter 1, large areas in the western part of the Netherlands have a subsoil with an extremely low bearing capacity. Examples are the peat areas marked in Figure 1.1 and many stretches along the rivers. Such subsoils are sensitive to settlement and potentially problematic with respect to the construction of road infrastructure.

The application of traditional, relatively heavy materials, e.g. sand, in a sub-base causes (uneven) settlements of the roads constructed in these areas with a weak subsoil. It is however possible to reduce the extent of settlements by using adequate light-weight materials in the sub-base. In this way the additional load on the subsoil can be reduced or even eliminated.

In the next sections, problematical aspects related to subsoils with a poor bearing capacity and alternative pavement structures on poor subsoils are reviewed and discussed. Firstly, pavement structures others than those with light-weight sub-base materials are described. Secondly, a general description of pavement structures with light-weight materials is given. Section 2.3 deals with light-weight sub-base materials themselves, classified into three groups: granular materials, light-weight concrete types and plastic foams. Finally some concluding remarks with respect to the applicability of the reviewed light-weight sub-base materials are made.

2.2 PAVEMENT STRUCTURES ON A SUBSOIL WITH POOR BEARING CAPACITY

The presence of compressible subsoils such as peat or clay in certain areas (often located along rivers and in polder areas) implies the construction of road embankments on a subsoil that is sensitive to settlements. Access embankments to bridges [1, 2] and widening of existing roads [3, 4] are other examples of situations which sometimes dictate the construction of road embankments on a compressible subsoil.

2.2.1 Problematical issues related to poor bearing subsoil

As indicated in section 2.1 the major problem in road constructions on subsoils with low bearing capacity is the occurrence of settlements. The settlements of a road reduce its serviceability. The problem is underlined especially when adjacent structures, i.e. houses, garages, bridges, are founded on piles and thus are not subjected to settlements. Levelling works are regularly needed to overcome the height differences between settlement-free buildings and surrounding pavements on the subsoil with low bearing capacity. The levelling layers, however, result in new substantial settlements due to extra loading of the weak soil and the problem returns after a certain period of time.
If thick compressible peat or clay layers are present in the subsoil the settlement process caused by overloading consists of two components. The first component, so-called primary settlements, are caused by consolidation and occur within a relatively short period of time. The second component regards long-term secondary settlements, also called 'creep'.

2.2.2 Alternative pavement structures on poor bearing subsoil

Apart from reducing the dead weight of the road structures (see subsection 2.2.3), the settlements and stability problems on weak subsoils can be solved in three other ways. The subsoil could be improved, replaced, or the road structure could be founded on piles if the subsoil layers of bad quality are too thick.

Improvement of compressible subsoils can be done by simply putting a surcharge on it, enabling the settlements to take place before constructing the upper pavement layers. This approach is very time consuming. The consolidation phase of the settlement process, however, can be accelerated by vertical drains which enable the drainage of groundwater to level the excessive pore pressures. Sometimes either chemical treatments or soil reinforcement lead to a significant increase of the subsoil bearing capacity.

In order to prevent future settlements it is also possible to replace problematical compressible subsoils by sand or another adequate material but it is considered to be a radical measure. Subsoil excavation and its replacement is only economical feasible in case of relatively small quantities.

The most expensive way of avoiding settlement and stability problems related to soft subsoil is foundation of the road structure on piles. Either reinforced or pre-stressed concrete plates, which bear the pavement structure, are laid on piles driven-in to the depth of incompressible soil layers. The piles are most of the time made of concrete but there are also projects with wooden piles [5]. The length of the piles can exceed 10 m depending on the local subsoil profile. Generally speaking, the method is extremely expensive and only applied on short road sections in areas with extremely difficult soil conditions.

2.2.3 Pavement structures with light-weight sub-base materials

The use of light-weight fill and foundation materials instead of the traditional 'heavy' materials enables the road construction engineer to reduce or even eliminate the additional load on the subsoil. The light-weight materials in some cases allow us to construct pavement structures (including embankments) with a total weight that does not exceed the weight of the excavated subsoil. In this case the natural weight balance is not disturbed and no settlements will occur. In Dutch road engineering practice, however, predominantly settlement-poor and only rarely settlement-free pavement structures have been constructed.
2.3 LIGHT-WEIGHT SUB-BASE MATERIALS

Materials with a density lower than the density of ordinary subsoil in wet condition are considered as light-weight sub-base materials. In literature, e.g. in [6], a density of $\rho = 1700 \text{ kg/m}^3$ is assumed to be an appropriate value for a subsoil in wet conditions. In engineering practice a subsoil with poor bearing capacity appears to have a significantly lower density in saturated state than the above-mentioned limit value, namely approximately 1100 kg/m$^3$. However, not only the peat has to be replaced but also a pavement structure consisting of materials with a density of 1700 to 2400 kg/m$^3$ has to be built. As a rule the maximum density of light-weight material is taken as approximately 1000 kg/m$^3$. In the following subsections an inventory of the sub-base materials which fulfil the last mentioned, more convenient density criterion will be made and their suitability commented.

2.3.1 Inventory

Three different kinds of light-weight sub-base materials can be distinguished that are widely available and frequently applied in road engineering practice. Those materials are:

a) light-weight granular material
b) light-weight portland cement concrete
c) plastic rigid foam

Also some organic (wood fiber) or waste materials (shredded/unshredded tires, plastic waste materials) are applied but their use could be suitably qualified as experimental mainly aimed to create a possibility for dumping of waste materials locally available in significant quantities. Since these materials are only used in exceptional situations they are considered beyond the scope of this study.

2.3.2 Light-weight granular materials

Unbound materials which can be considered as light-weight materials, are:

a) flugsand
b) expanded clay

Flugsand is a porous material with a volcanic origin. In Europe it is excavated in the Eifel in Germany. Its in-situ density varies between 840 and 1050 kg/m$^3$ [6]. The material is very porous and, if laid in a humid environment under groundwater level, it absorbs water to a great extent which reduces its applicability as a light-weight sub-base material. Furthermore, compaction during road construction contributes to the increase
of the original density as well. A good characteristic of flugsand is its high angle of internal friction (shear strength) and stability caused by the angular grain shape. The elasticity modulus of flugsand, based on the results of compression tests on cylindrical specimens [7], has values between 20 and 50 MPa dependent on the stress conditions in the specimens.

Another material with an appropriate density, shear strength and resistance to compressibility to be used as light-weight fill material is expanded clay. The production process consists of mixing clay from various layers and adding air void forming additives to the basic material. The next step is exposure of the clay to high temperature (1100°C [7]) which results into expansion of the clay grains. The final product, expanded clay grains, is characterized by a high porosity and a closed cell structure. The dry density of the material when used as a layer ranges between 600 and 650 kg/m^3 but, because expanded clay grains absorb a significant quantity of water (up to 40% v/v), their wet density goes up to 1050 kg/m^3 [8]. As a light-weight fill material mainly the fraction 10/16 mm is used. Because of the uniform grain size the angle of internal friction of expanded clay is 35°. The elasticity modulus (assuming this term can be used for expanded clay grains) has a value lower than 20 MPa under sub-base stress conditions.

In the literature, e.g. [6] and [9], also different types of fly ashes and foamed slags are mentioned as light-weight fill/foundation materials. However, the use of these unbound materials in sub-bases does not lead to substantial reduction of the load of road embankments on soft compressible subsoil. Due to their insufficiently low density neither fly ashes nor foamed slags are considered further.

2.3.3 Light-weight concrete types

Common portland cement concrete has a density of about 2400 kg/m^3. Under light-weight concretes suitable for light-weight sub-base purposes we understand the concrete types with \( \rho \leq 1000 \) kg/m^3. Such a low concrete density is possible by (partly or totally) replacing the heavy aggregate by air and/or by increasing the porosity of the cement paste. The air can be present in a light-weight concrete either inside closed cells uniformly distributed in the mortar, or inside the pores of porous aggregate.

Light-weight concrete types with a cellular structure are foamed concrete and aired concrete. The major difference between these two types of light-weight concretes is the way in which the air voids have been formed inside the material. The air cells in the aired concrete are formed as a consequence of a chemical reaction. In foamed concrete the air cells are formed by mixing pre-formed foam and cement slurry just before casting. The shortcoming of aired concrete, which makes application as a sub-base material impossible is that, contrary to foamed concrete, the hardening of the aired concrete occurs in autoclaves under high pressure and temperature. Foamed concrete is a suitable sub-base material widely used in engineering practice, particularly because it can be produced in situ. Its properties will be discussed in Chapter 4 together with the results of our
material characterization tests on specimens of a very light-weight foamed concrete with a dry density of approximately 500 kg/m$^3$.

2.3.4 Plastic foams

Plastic foams used in geotechnical applications, e.g. in road embankments, nowadays are also called geofoams. This generic name introduced by Horvath [10] suggests the affiliation of plastic foams to geosynthetic products (the other geosynthetics are geotextiles, geomembranes and geogrids). The principal difference between geofoams and other light-weight fill/foundation materials is their much lower density. Among various related products only two types of plastic foam: extruded polystyrene (XPS) and expanded polystyrene foam (EPS), fulfill the needs. They are both affordable and have proper geotechnically related properties to be used in road construction.

EPS is an ultra light-weight sub-base material with a dry density between 15 and 35 kg/m$^3$. The extremely low density combined with low water absorption, high durability and appropriate mechanical properties contributes to a continuously increasing popularity of EPS for large-scale applications in areas with a compressible subsoil. The use of EPS enables highway engineers to design and construct real settlement-free road embankments under very difficult subsoil conditions. Details regarding the production process, the results of performed materials research on EPS geofoam and the analysis of various pavement structures with an EPS sub-base, are the subjects of the essential chapters of this thesis starting with Chapter 5.

XPS is manufactured by expanding the polystyrene in different densities. The geotechnically relevant properties of XPS, like those of EPS, are correlated to the material density. Despite its very low dry density, 20 to 50 kg/m$^3$, XPS as a light-weight sub-base material is applied to a limited extent for two reasons. Firstly, because of its blowing agent - chlorofluorocarbon gas used for expanding the polystyrene to the cellular product XPS. This gas is linked to depletion of the ozone layer. Secondly, the manufacturing process of XPS limits the basic products to relatively thin panel- and plank-shaped pieces. The limitation of the maximum length is an obstacle for the application of XPS for fill/foundation purposes where large blocks are desirable.

2.4 CONCLUDING REMARKS ON LIGHT-WEIGHT SUB-BASE MATERIALS

The purpose of the use of light-weight fill/foundation materials is to obtain a significant reduction of the dead weight of a pavement structure in a cost effective way. Having further in mind that the representative wet density of compressible soil is 1100 kg/m$^3$ there are actually only two light-weight materials which can be used on a large scale. These materials are light-weight types of foamed concrete and expanded polystyrene
(EPS) geofoam. In following chapters relevant aspects related to foamed concrete and EPS will be discussed in more detail.

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CHAPTER 3

REPRESENTATIVE SUB-BASE CONDITIONS

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3.1 INTRODUCTION

In a pavement structure the layer between the roadbase and the subgrade is called the sub-base. The function of the sub-base is twofold: to transfer the load from the roadbase to the subsoil and to protect the subsoil against the damaging influence of frost. One can say that the practical purpose of the sub-base is to enlarge the thickness of the pavement structure in the most economical way. An EPS sub-base or a sub-base of foamed concrete is sometimes applied, instead of a sand sub-base, to reduce the total weight of a pavement structure. Because of the loadspreading in the upper layers, a passing vehicle causes only low stresses in the sub-base. When EPS blocks are applied these stresses need to be low indeed, otherwise it would be impossible to use a low modulus material such as EPS in the sub-base. The location of the sub-base material in the pavement structure implies that its material is subjected to varying moisture and temperature conditions.

As pointed out in chapter 2 the modulus of elasticity of EPS is much lower than the modulus of elasticity of traditional sub-base materials. Furthermore, the thermal conductivity of EPS is also much lower (factor 16) than that of sand. As a material with such different characteristics, EPS influences the loadspreading and heat flow in a pavement structure in general, and in the sub-base in particular. Therefore, the representative temperature and stress values occurring in the EPS sub-base cannot be assumed to be equal to those occurring in a sand sub-base. In the case of foamed concrete, the second most important light-weight sub-base material, the above-mentioned differences with traditional materials are much less significant. In the following sections the temperatures, moisture contents and stresses in sub-bases of pavements with an EPS sub-base will be discussed.

3.2 TEMPERATURE DISTRIBUTION

Knowledge about the temperature distribution in asphalt pavement structures is of great importance, as the temperature determines for example the stiffness of the asphalt layers, the frost penetration depth and the pavement surface temperature. Generally spoken, the asphalt layers (wearing course and base course) are much more sensitive to temperature than the materials used in the roadbase and the sub-base. Since the stiffness of bitumen is dependent on the temperature, obviously the temperature will strongly in-

Figure 3.1 - Cross section of a road construction with the EPS sub-base
fluence the asphalt mix properties, and through this the horizontal strain due to traffic loads which is usually the dominant design criterion for asphalt pavement structures.

The minimum required total thickness of the pavement structure must be greater than the frost penetration depth. If this is not the case then frost heave and especially thaw can lead to a serious loss of stability of the subsoil, resulting in severe damage of the pavement structure. Knowledge of the frost penetration depth in pavement structures is also of importance in case of insertion of water and gas pipelines in the subsoil. The frost must not appear in the surrounding material of the pipelines.

The temperature at the pavement surface is related to the safety of driving on the road under winter conditions.

The presence of EPS geofoam will cause a different temperature distribution in road pavement structures. In order to find out to which extent the use of EPS affects the temperature distribution, we have carried out finite element calculations on several asphalt pavement structures with either a sand or an EPS sub-base. The calculations were based on the Fourier's differential equation derived from the equation for the energy balance of an indefinite small element. The solution of this equation is the temperature distribution in space and time. On the basis of the physical model of heat balance relations can be found between the measurable climatic data (air temperature, wind-speed, humidity, cloud cover, global radiation) and the boundary condition for Fourier's equation at the pavement surface. The theoretical background and finite element analysis results are discussed in the next subsections.

3.2.1 Heat transfer directions

In reality, there are two opposite situations: a temperature of the atmosphere higher than the temperature of the subsoil under the pavement structure, and a subsoil warmer than the surrounding air.

In the first case the heat transfer is in the downward direction. The daily temperature variations decrease also with the depth. The depth of the heat penetration into the subsoil and the amplitude of the daily and seasonal temperature variations depend on the thermal properties of the soil. The daily penetration is in the order of 0.3 to 0.8 m, while the annual penetration of the heat into the subsoil is about 4 m.

In the second case the temperature of the upper layers of the pavement structure is lower than the temperature of the lower layers. The direction of the heat transfer is upwards. This is the consequence of the geothermal heat flux caused by the heat flow from the interior of the earth. Conventionally, in the Netherlands, a temperature value of 10°C (which is equal to the mean annual air temperature) is taken as a constant temperature beneath the depth of annual heat penetration in the subsoil.
3.2.2 Mechanisms of heat transfer

The temperature distribution in a pavement structure depends on 1) the heat exchange at the road surface and 2) the heat transfer through the pavement structure and the subsoil. The mechanisms of these phenomena are complex. Figure 3.2 [1] shows the heat balance at the pavement surface.

The contribution of the following transport mechanisms is the most important for the heat exchange at the pavement surface:

a) global radiation (short-wave radiation)

b) effective outgoing radiation (long-wave radiation)

c) heat flow by convection

Ad. a The positions of short-wave radiation which reach the road surface are called global radiation. Global radiation consists of direct solar radiation and diffuse radiation (one part of dispersed radiation). The factors that influence the intensity of the global radiation at the pavement surface are: sun’s altitude, cloud cover, season, position on earth, the orientation of the surface and pollution of the atmosphere.

Ad. b The pavement surface, when heated by the absorption of solar radiation, becomes a source of long-wave infrared radiation. The atmosphere, in return, reradiates the absorbed radiation partly to space and partly to the road surface (counter radiation). The effective outgoing radiation from the pavement surface is equal to the total long-wave energy sent from the surface minus the counter radiation from the atmosphere.

Ad. c The heat flow by the convection in the air (sensible heat) is caused by wind and the changes in the air density with temperature. If the pavement surface temperature is higher than the temperature of the air which is in contact with the surface, this will result in an upward heat flow. In principle, sensible heat is a function of the temperature difference between the pavement surface and the wind velocity.
Representative Sub-base Conditions

Within the pavement structure and the subsoil underneath it, conduction is the predominant mechanism of heat transfer. Heat conduction is dependent on the following pavement material and subsoil properties: thermal conductivity, density and heat capacity per unit volume. These pavement material and soil properties are dependent on the temperature and the degree of saturation of them.

The thermal conductivity \( \lambda \) of EPS foam has values from 0.031 to 0.036 W/mK [2] and it increases by about 4% for each 1% by volume water absorption. The EPS sub-base lies in a moist environment. As the representative \( \lambda \)-value 0.036 W/mK is chosen. The specific heat of EPS20 at 22°C is 1210 [J/kgK] and at 5°C it is 1130 [J/kgK] [2]. The used specific heat of the EPS foam is 1160 [J/kgK].

### 3.2.3 Modelling of temperature distribution

The temperature distribution in a solid medium is a function of time and space. It is derived from a partial differential equation which is based on the equation for the energy balance of an indefinite small element.

#### 3.2.3.1 Fourier's differential equation

The complete partial differential equation (so-called Fourier's differential equation) for the heat transfer by conductivity through a solid element is:

\[
\frac{\delta T}{\delta t} = a \left( \frac{\delta^2 T}{\delta x^2} + \frac{\delta^2 T}{\delta y^2} + \frac{\delta^2 T}{\delta z^2} \right) \quad [K/s] \quad [eq. 3.1]
\]

where

- \( a = \frac{\lambda}{\rho c} \) - the coefficient of thermal diffusion \([m^2/s]\)
- \( \lambda \) - thermal conductivity of material \([W/mK]\)
- \( \rho \) - density \([kg/m^3]\)
- \( c \) - specific heat of material \([J/kgK]\)

The model for the heat transfer in the pavement structure is one-dimensional. It is assumed that the horizontal temperature gradient is absent, therefore the temperature distribution in the horizontal direction is constant. With respect to the heat transfer at the road sideslides only the heat transport in the vertical direction is considered. For one-dimensional heat transport the equation [eq. 3.1] is reduced to:

\[
\frac{\delta T}{\delta t} = \frac{\lambda}{\rho c} \frac{\delta^2 T}{\delta z^2} = a \frac{\delta^2 T}{\delta z^2} \quad [K/s] \quad [eq. 3.2]
\]

where

- \( z \) - depth \([m]\)
Representative Sub-base Conditions

In order to solve the partial differential equation [eq. 3.2] it is necessary to define two boundary conditions:

1) The first boundary condition states that the heat transfer is irrelevant at great depth in the subsoil because the temperature is constant (in the Netherlands 10°C) at a depth of more than about 4 m.

2) The second boundary condition states that at the surface of the pavement structure the heat flux from the surface into the pavement structure \( q_{in} \) in [eq. 3.3]) balances the absorbed sun radiation and the convective and long-wave radiation doses.

\[
q_{in} = -\lambda \frac{\delta T}{\delta z} \quad [W] \quad [eq. 3.3]
\]

where \( q_{in} \) - heat flux \([W/m^2]\)

\( \delta T/\delta z \) - the temperature gradient in the vertical direction \([K/m]\)

The heat flux is the heat flow through a unit cross-sectional area. The heat flow, and the heat flux are positive and oriented downward when the temperature decreases with the depth.

### 3.2.3.2 Model of heat transfer

The second boundary condition [eq. 3.3] contains absorbed sun radiation, convective and long-wave radiation flux and the heat flux \( q_{in} \) at the pavement surface. To calculate this value the heat transfer model has been taken which is showed in Figure 3.3. By this model of heat transfer at the pavement surface the multitude of heat transfer mechanisms (Figure 3.2) is reduced to the four mechanisms that are dominant. As described in subsection 3.2.2, from the four main heat mechanisms for heat exchange at the road surface three occur in the surrounding atmosphere. They are: 1) the global radiation at the pavement surface \( q_L \); 2) effective outgoing radiation from the pavement surface \( q_S \) and 3) the heat flow by convection (sensible heat) between the pavement surface and the air \( q_c \). The fourth mechanism occurring in the subsurface is: 4) heat flow by conduction through the pavement structure \( q_{in} \). The model in Figure 3.3 may be represented symbolically by the heat balance equation at the pavement surface:

\[
q_{in} = A q_z - (q_S + q_c) \quad [W/m^2] \quad [eq. 3.4]
\]

where \( A \) - coefficient of absorption of the pavement surface \([-]\)

\( q_z \) - measured global radiation at the pavement surface per square meter \([W/m^2]\)
Representative Sub-base Conditions

$q_e$ - effective outgoing radiation from the pavement surface per square meter \[W/m^2\]
$q_c$ - heat flow by convection (sensible heat) between the pavement surface and the air per square meter \[W/m^2\]

The pavement surface is commonly assumed to emit and absorb energy as a gray body in infrared region. The flux of radiation is directly proportional to the forth power of its absolute temperature. Brunt found the empirical relation for the counter radiation [3]. So the net effective outgoing long-wave radiation flux can be written as [3]:

$$q_s = \varepsilon \sigma T_s^4 - \varepsilon [(a + b\sqrt{p}) \sigma T_a^4 + 60n)] \quad [W/m^2] \quad \text{eq. 3.5}$$

where  
\(\varepsilon\) - surface emissivity [-]
\(\sigma\) - radiational constant in Stefan's law = 5.67 \(10^8\) [W/m\(^2\)K\(^4\)]
\(T_s\) - surface temperature [K]
\(a\) - empirical constant = 0.53 [-]
\(b\) - empirical constant = 0.067 [hPa\(^{1/6}\)]
\(p\) - vapour pressure [mbar]= [hPa]
\(T_a\) - air temperature [K]
\(n\) - cloud cover [-]

The air convection may be taken into account by means of the following expression for the heat flow by convection (sensible heat) between the pavement surface and contact air:

$$q_c = \alpha_c (T_s - T_a) \quad [W/m^2] \quad \text{eq. 3.6}$$

where  
\(\alpha_c\) - coefficient of heat transfer by air convection [W/m\(^2\)K]

The heat transfer through the air by convection is different under the two qualitatively different circumstances. If the temperature increases with height the conditions are assumed stable. If temperature decreases with height the conditions are assumed unstable.

The established empirical relation for the global radiation at the surface, for the outgoing radiation from the pavement surface and for the heat flow by convection between the pavement surface and contact air enables the calculation of the values for the heat flux \(q_{sb}\) by using the expression [eq. 3.4]. In that way the second boundary condition [eq. 3.3] becomes known.

3.2.3.3 Analytical solution

An analytical solution of the Fourier’s partial differential equation [eq. 3.2] is available only in special cases, namely when instead of the actual pavement structure a uniform half space is assumed and when the pavement surface temperature is a periodical function of time.
Representative Sub-base Conditions

By assuming a cosine function for the pavement surface temperature, this temperature could be given as:

\[ T(z=0, t) = T_{\text{mean}} + T_0 \cos(\omega t) \quad [K] \]  
\[ \text{[eq. 3.7]} \]

where
- \( T_{\text{mean}} \) - constant temperature at great depth in the subsoil \([K]\)
- \( T_0 \) - amplitude of the temperature at the pavement surface \([K]\)
- \( \omega \) - angular frequency \([s^{-1}]\)
- \( t \) - time \([s]\)

Taking the pavement surface temperature according to equation [eq. 3.7] the solution of the Fourier’s partial differential equation [eq. 3.2] is expressed by:

\[ T(z, t) = T_{\text{mean}} + T_0 e^{-Az} \cos(\omega (t - \frac{Z}{v})) \quad [K] \]  
\[ \text{[eq. 3.8]} \]

where
- \( A = \frac{\omega}{v} = \sqrt{\pi / a T_p} \) - measure for the damping \([m^2]\)
- \( a \) - the coefficient of thermal diffusion \([m^2/s]\)
- \( T_p \) - period of the heatwave \([s]\)
- \( \omega = 2 \pi f = 2 \pi / T_p \) angular frequency \([s^{-1}]\)
- \( f \) - frequency \([s^{-1}] \equiv [Hz]\)
- \( v \) - speed of the heatwave propagation in positive z direction \([m/s]\)

This solution [eq. 3.8] can be used for the calculation of the temperature distribution in soil as a result of daily and annual temperature variations. However, a pavement structure with a few layers of different materials can not be approximated with a uniform half space. Also the cosine function [eq. 3.7] is not representative for the real surface temperature. Thus, a general analytical solution of the Fourier’s partial differential equation [eq. 3.2] useful for pavements does not exist.

The temperature distribution in a multilayer system can be found by a numerical approximation of equation [eq. 3.2]. The finite element (FE) program WEGTEM [4] computes roughly this solution when the boundary condition [eq. 3.3] is assumed. In that way the temperature in a pavement structure as a function of time and place can be closely approximated.

3.2.4 The finite element program WEGTEM

The FE program WEGTEM is designed for one-dimensional calculations of the nonstationary temperature distribution in a multilayer system. The thermal boundary conditions at surface of the top layer represent the influence of the main heat transfer mechanisms in the atmosphere. The boundary condition under the bottom layer implies the influence of the geothermal heat flux represented by a constant temperature (10°C) at a certain depth (in this thesis taken as 4 m [5]). The program computes the Fourier’s partial differential equation [eq. 3.2]:

EPS AS A LIGHT-WEIGHT SUB-BASE MATERIAL IN PAVEMENT STRUCTURES
\[ \rho c \frac{\delta T}{\delta t} = \lambda \frac{\delta^2 T}{\delta z^2} \quad \text{[W/m}^3] \quad \text{[eq. 3.8]} \]

The theoretical background of the finite element method, which is used for the calculation, is described in [6]. The multilayer system is divided into layers. The temperature distribution in every single element is linear. The number of elements in vertical direction is at least equal to the number of system layers. The temperature is not calculated for all points but only for the nodes between the elements. The nodes are placed at selected depths \( z_i \) in the multilayer system. Time is also discreted and the solution \( T(t) \) at the discrete points of time \( \Delta t \), \( 2\Delta t \), ..., \( n\Delta t \) is approximated by temperatures \( T^1 \), \( T^2 \), ..., \( T^n \).

### 3.2.4.1 Input data

Among the input data necessary for the FE calculations of temperature distribution there are two qualitatively different kinds of information. One group of data describes the multilayer system and the second group describes the climate. Besides that the period \( \Delta t \) between discrete points of time the initial moment, \( t_0 \), has to be defined.

The multilayer system is described by the number of layers and each of these layers is further subdivided into smaller elements (in vertical direction). The materials in the pavement layers and the subsoil have to be described by their thermal properties. The thermal material properties which WEGTEM takes into account, are:

- coefficient of absorption of the pavement surface \( A \) [-]
- density \( \rho \) [kg/m\(^3\)]
- specific heat of material \( c \) [J/kgK]
- thermal conductivity of material \( \lambda \) [W/mK]

The model in Figure 3.3 shows the main mechanisms of heat transfer at the pavement surface. The heat flow from the atmosphere can be calculated by using the following information:

- air temperature at height of 1.5 m \( T_a \) [°C]
- surface temperature \( T_s \) [°C]
- wind-speed at height of 10 m \( u_{10} \) [m/s]
- relative humidity \( \Phi \) [-]
- global radiation at the pavement surface per square meter \( q_z \) [W/m\(^2\)]
- cloud cover \( n \) [-]

### 3.2.4.2 Output data

The finite element program WEGTEM offers the following data for a period of one year in his standard output:
Representative Sub-base Conditions

- Percentage of total considered period of time that the temperature in the centre of each layer is within each temperature class (temperature intervals of 5°C).
- Percentage of total considered period of time that the temperature gradient in the system layers has a value within each temperature gradient class (intervals of 0.04°C/mm).
- Period of time (hours) that the surface temperature belongs to selected temperature classes.
- Daily temperature amplitudes in the system layers with frequency of appearance.
- Duration of frost (hours) at selected depth classes.

For a period of one week the temperatures can be obtained for each hour in all nodes. The temperature at the chosen nodes is graphically presented for the week under consideration. The temperature distribution at any moment in this week is constructed by connecting discrete temperature values in the nodes of the total pavement structure.

The most important information for asphalt pavements are the periods of time that the distinguished temperature values are present in the three asphalt layers: dense asphalt concrete (d.a.c.), open asphalt concrete (o.a.c) and gravel asphalt concrete (g.a.c).

3.2.5 Analyzed pavement structures and simulated climatic conditions

One-dimensional calculations of the temperature distribution were carried out for 54 asphalt pavement structures, half of them having a sand sub-base and half of them having an EPS sub-base. Each pavement structure with a sand sub-base has a corresponding pavement structure with expanded polystyrene foam (EPS).

For the layer material properties the representative values are the same for all analyzed pavements. Data about climatic conditions were registered during a whole year with both a moderate summer and winter typical for Dutch climatic conditions.

3.2.5.1 Modelled pavement structures

The asphalt pavement structures are chosen such that the thickness of all the layers varied from very thin to very thick. It is assumed that in this way enough different pavements structures are considered for general conclusions. The pavement structures and the layer thicknesses are shown in Figure 3.4.

The first boundary condition says that the heat transfer is irrelevant at a depth of 4 m below the pavement surface. At this depth the temperature is constant and equal to 10°C. For this reason the total thickness of the multilayer systems is limited to 4 m.

Figure 3.4
pavement structures with a sand or EPS sub-base
The asphalt total thickness consists in reality of two or three separate layers. If e.g. \( h_A = 0.1 \text{ m} \) there is a dense asphalt concrete layer (d.a.c.) and a gravel asphalt concrete layer (g.a.c.) of 50 mm each. If \( h_A \) is equal to 0.2 or 0.3 m then an open asphalt concrete layer (o.a.c.) is laid between the d.a.c. and g.a.c. layers. In that case the pavement structure is as follows: 40 mm d.a.c. as toplayer (wearing course), 40 mm o.a.c. and the rest is g.a.c. (120 mm or 220 mm). The layers of the model correspond with the asphalt sublayers.

### 3.2.5.2 Thermal material parameters

The information in the literature about the thermal properties of pavement materials and soil is not uniform. Thermal properties are dependent on temperature, water content, movability of this water etc. For analysis purposes it is assumed, however, that each thermal property of each material is a constant (see Table 3.1 [1, 2, 7, 8, 9, 10]).

<table>
<thead>
<tr>
<th>Material</th>
<th>( \rho ) [kg/m(^3)]</th>
<th>( c ) [J/kg K]</th>
<th>( \lambda ) [W/m K]</th>
<th>( a ) [m(^2)/s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>asphalt</td>
<td>2200</td>
<td>800</td>
<td>1.20</td>
<td>6.82E-7</td>
</tr>
<tr>
<td>roadbase</td>
<td>2040</td>
<td>840</td>
<td>0.52</td>
<td>3.03E-7</td>
</tr>
<tr>
<td>sand</td>
<td>1750</td>
<td>960</td>
<td>0.58</td>
<td>3.45E-7</td>
</tr>
<tr>
<td>EPS</td>
<td>20</td>
<td>1150</td>
<td>0.036</td>
<td>15.52E-7</td>
</tr>
<tr>
<td>peat</td>
<td>1100</td>
<td>2870</td>
<td>0.50</td>
<td>1.58E-7</td>
</tr>
</tbody>
</table>

Table 3.1 - Thermal parameters of pavement materials and subsoil

For peat the parameters are chosen for a degree of saturation of maximal one (± 80% volume of water). This is the common state of peat in Holland. When air is displaced by water the thermal conductivity of peat increases. The reason is that the conductivity of water is 22 times greater than that of air. It a means faster heat conduction through saturated peat. But a big specific heat of saturated enables the storage of a lot of convected heat in this layer. Therefore saturated peat contributes to the retardation of the heat transfer in a pavement structure.

### 3.2.5.3 Climatic conditions

The climatic conditions are represented by the measurement results of the meteorological station Den Helder. The information mentioned in paragraph 3.2.4.1 required as input has been collected during the whole of 1981. The summer and the winter in 1981 have been without extreme temperatures. Also other conditions were quite normal and therefore the 1981 data are suitable as average climatic conditions for the calculation of the temperature distribution in pavement structures in the Netherlands. One might
argue that extreme conditions would be more interesting but unfortunately data for such conditions were not available.

### 3.2.6 Calculation results

In the following paragraphs the results of WEGTEM finite element calculations into the temperature distribution in asphalt pavement structures with a sand sub-base and an EPS sub-base are presented.

#### 3.2.6.1 Temperature in asphalt layers

The percentage of time that a specific temperature in the middle of the asphalt layers is exceeded is graphically presented in Figures 3.5 to 3.8 for two couples of selected pavements. The first two pavements are thin structures (Fig. 3.5 and 3.6) while the second couple of pavements have the greatest asphalt layer and roadbase thickness (Fig. 3.7 and 3.8). These two couples of asphalt pavement structures are illustrative for all 27 couples of similar pavement structures.

In the upper asphalt layers (d.a.c. and o.a.c.) the temperature variation is bigger than in the lower layer (g.a.c.). The temperature trajectory within the g.a.c. is smaller due to damping of the heat wave in the d.a.c. and o.a.c. layers.

Mutual comparisons of the presented graphs with and the graphs of all the other analyzed asphalt pavement structures, show a very small difference between the cumulative temperature distributions in pavement with and without EPS. Therefore, the general conclusion is that using EPS foam as a sub-base material in asphalt pavement structures instead of sand does not cause substantial temperature differences in the asphalt layers.

![Graph showing temperature distribution](image)

**Figure 3.5**

Cumulative temperature distribution in the asphalt layers during 1981 in the asphalt pavement structure with: 0.05 m d.a.c., 0.05 m g.a.c., 0.1 m sand and 3.4 m EPS
Representative Sub-base Conditions

**Figure 3.6**
Cumulative temperature distribution in the asphalt layers during 1981 in the asphalt pavement structure with: 0.05 m d.a.c., 0.05 m g.a.c. and 3.5 m sand

**Figure 3.7**
Cumulative temperature distribution in the asphalt layers during 1981 in the asphalt pavement structure with: 0.04 m d.a.c., 0.04 m o.a.c., 0.22 m g.a.c., 0.3 m roadbase, 0.1 m sand and 2.0 m EPS

**Figure 3.8**
Cumulative temperature distribution in the asphalt layers during 1981 in the asphalt pavement structure with: 0.04 m d.a.c., 0.04 m o.a.c., 0.22 m o.a.c., 0.3 m roadbase and 2.1 m sand
Representative Sub-base Conditions

3.2.6.2 Temperatures at the interface layers in time

According to the FE calculation results, the temperature distribution in the asphalt pavement structure with an EPS sub-base is somewhat different from that in a structure with a sand sub-base. The annual maximum surface temperature is higher, the minimum surface temperature is lower, the frost penetration is greater in (case of) an EPS sub-base. On the contrary, the duration of a certain temperature in the asphalt layers is almost the same. The explanation of this phenomenon requires an insight into the temperature distribution along the vertical axis through the pavements.

Graphs of the temperature distribution are valid for one fixed moment in time. One hour later the temperatures at respective positions in pavements have other values. Representative moments are those when the temperatures have extreme values. An analysis of the temperature distribution at these moments takes into account the most unfavorable circumstances.

The Figures 3.9 and 3.10 show the temperatures in two asphalt pavement structures for the coldest 8 days of the considered year. The first pavement (Figure 3.9) is with an EPS sub-base, the second one (Figure 3.10) has a sand sub-base. The temperature lines drawn apply to the pavement surface and layer interfaces. The temperature lines show two extreme moments: in the night of February 23 at 06h the surface temperature has reached the absolute minimum for 1981, while at February 28 at 13h the highest surface temperature within the period under consideration (February 21 to 28) occurred.

Fig. 3.9 - Temperature at the pavement surface and layer interfaces during the coldest period in the winter of 1981 (February 21 to 28); asphalt pavement structure: 0.05 m d.a.c., 0.05 m g.a.c., 0.15 m roadbase, 0.1 m sand and 2.0 m EPS sub-base
Representative Sub-base Conditions

Fig. 3.10 - Temperature at the pavement surface layer interfaces during the coldest period in the winter of 1981 (February 21 to 28); asphalt pavement structure: 0.05 m d.a.c., 0.05 m g.a.c., 0.15 m roadbase and 2.1 m sand sub-base

For the two previously analyzed asphalt pavement structures Figure 3.11 and 3.12 present the progress of the temperature during the warmest 8 days in August 1981. On August 5 at 12\textsuperscript{th} there was the absolute maximum temperature in 1981.

Fig. 3.11 - Temperature at the pavement surface and layer interfaces during the warmest period in the summer of 1981 (August 1 to 8); asphalt pavement structure: 0.05 m d.a.c., 0.05 m g.a.c., 0.15 m roadbase, 0.1 m sand, 2.0 m EPS sub-base
Fig. 3.12 - Temperature at the pavement surface and layer interfaces during the warmest period in the summer of 1981 (August 1 to 8); asphalt pavement structure: 0.05 m d.a.c., 0.05 m g.a.c., 0.15 m roadbase and 2.1 m sand sub-base

3.2.6.3 Temperature distribution

The calculation of the temperatures in all the nodes of the multilayer systems enables the determination of the temperature distribution in the asphalt pavement structures under consideration at fixed moments in 1981 with extreme climatic conditions. In Figure 3.13 the temperature distributions in two thin pavement structures for the absolute minimum temperature in 1981 are shown. The temperature lines belong to the pavement with an EPS sub-base and the corresponding pavement structure with a sand sub-base.

In a similar way in Figure 3.14 the temperature distribution for the same thin pavement structures at the warmest moment in 1981 is shown.

Figures 3.15 and 3.16 present temperature values in two thicker pavement structure (with a 0.15 m thick roadbase) at the fixed moments with extreme temperatures in the year under consideration.
Figure 3.13

Temperature distribution at the absolute coldest moment in 1981 in the asphalt pavement structure with: 0.05 m d.a.c., 0.05 m g.a.c., 0.1 m sand and 2.0 m EPS/sand sub-base

Figure 3.14

Temperature distribution at the absolute warmest moment in 1981 in the asphalt pavement structure with: 0.05 m d.a.c., 0.05 m g.a.c., 0.1 m sand and 2.0 m EPS/sand sub-base
Representative Sub-base Conditions

**Figure 3.15**
Temperature distribution at the absolute coldest moment in 1981 in the asphalt pavement structure with: 0.05 m d.a.c., 0.05 m g.a.c., 0.15 m roadbase, 0.1 m sand and 2.0 m EPS/sand sub-base.

**Figure 3.16**
Temperature distribution at the warmest moment in 1981 in the asphalt pavement structure with: 0.05 m d.a.c., 0.05 m g.a.c., 0.15 m roadbase, 0.1 m sand and 2.0 m EPS/sand sub-base.
Inspection of these graphs shows that in asphalt pavement structures with an EPS sub-base obviously a different temperature distribution occurs than in pavement structures with a sand sub-base. But the temperatures in the asphalt layers are almost identical in both types of pavement structures. Under extreme climatic conditions, the temperature in the asphalt of a structure with an EPS sub-base is less than 1°C different from the asphalt temperature in the corresponding structure with a sand sub-base. Practically spoken, using EPS foam in the sub-base has a negligible influence on the temperature in the asphalt layers. The explanation is that the EPS foam lies at such a great depth below the asphalt layers that the EPS foam insulating characteristics can not cause higher or lower temperatures in asphalt, where the temperature is only dependent on the climate. In contrast with this, the influence of the EPS sub-base is obvious in the roadbase. However, the materials used in the pavement layers below the asphalt are not sensitive to temperature, so EPS foam in road constructions does not have significant consequences for the life of asphalt pavement structures.

3.2.7 Temperatures occurring in an EPS sub-base

The maximum temperature calculated for the EPS layer of a pavement structure as a result of the climatic conditions considered was approximately 27°C. At the same time, the temperature at the pavement surface reached a level of 45°C. The lowest temperature in the EPS layer was -4°C, when the pavement surface temperature was equal to about -9°C. During the years with hot summers and cold winters in The Netherlands, or in other climatic areas, the extreme temperatures occurring in an EPS sub-base will be lower than -4°C and higher than 27°C. Accordingly, these limit temperatures define the minimum temperature interval the EPS samples should be exposed to. This is taken into account in the planning of experiments (subsection 5.3.4 and 5.6.4).

When applying an EPS sub-base, the frost penetration (which in the Netherlands rarely exceeds 1 m) will not reach below the sub-base. The use of the EPS foam will lead to extra good protection of the (peat) subgrade against low temperatures which will contribute to maintain the bearing capacity of the sub-soil in thaw period.

3.3 MOISTURE CONTENT

EPS is primarily applied on subsoils with a low bearing capacity, such as peat, where the groundwater level is frequently very high. In order not to disturb the existing natural weight balance, the subsoil is often excavated to a certain depth and replaced by EPS. The EPS built-in in the subsoil comes into contact with groundwater. Because the groundwater table can constantly change position, the EPS will be, at least temporarily, partly or entirely under the phreatic surface and if this state is of long duration EPS will absorb water. An eventual different mechanical behaviour of EPS in partly or entirely wetted state, in comparison with dry EPS, has to be taken into account in the
design procedure for pavement structures. For that reason water absorption tests have been performed as part of the material testing program (Chapter 5).

3.4 STRESS CONDITIONS UNDER DEAD WEIGHT AND TRAFFIC LOAD

EPS as a sub-base material is subjected to static and dynamic loads. Consequently, two stress components occur in EPS. Firstly, there is the static stress $\sigma_e$ caused by the dead weight of asphalt top layers and roadbase which lay above the EPS sub-base. Secondly, there is a dynamic stress $\sigma_d$ as a consequence of traffic loading. The frequency and duration of the dynamic loading are dependent, on the one hand, on the speed of the passing vehicle and, on the other hand, on the thickness and the stiffness of the upper layers. Initial calculations showed that the horizontal static and dynamic stress components in the EPS layer were so low that they were neglected in the material research program (Chapter 5).

The initial calculations showed that the representative maximum vertical stresses occurring in EPS were equal to about $\sigma_e = 20$ kPa (due to 1 m thick pavement layers laid above the EPS sub-base) and $\sigma_d = 35$ kPa (due to a 50 kN wheel load, diameter contact area 300 mm, on a concrete block pavement structure with a thin roadbase consisting of poor quality unbound materials) respectively. It should be noted that these values are on the high side.

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CHAPTER 4

FOAMED CONCRETE AS LIGHT-WEIGHT SUB-BASE MATERIAL

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4.1 INTRODUCTION

Foamed concrete was invented in Germany in 1920 [1]. Due to an imperfect production process and a variable quality it was used to a limited extent at that time. Nowadays however foamed concrete is a material with numerous applications. The presence of air cells in foamed concrete results not only in a relatively low density but in good acoustic and thermal insulation properties as well. All this, together with its reasonable stiffness, on-the-spot producibility, freedom in shaping and design and good workability constitute the reasons for the use of foamed concrete in terraced roofs, below houses and sport fields, in walls, bridge-foundations, roads, etc. Nowadays in The Netherlands approximately 100,000 m³ [2] of foamed concrete is used annually by the building industry, whereas in 1985 the annual quantity was about 70,000 m³ [3].

This chapter discusses foamed concrete as a light-weight sub-base material in pavement structures. Foamed concrete is a cement bound material with an air content ranging from 30 to 80% by volume [2]. Depending on the percentage of air cells inside, the density of foamed concrete mixes varies between 400 and 1600 kg/m³[4] (alternative upper limit is 1800 kg/m³ [5]). Due to its relative low (dry) density, foamed concrete is considered as a light-weight sub-base material. Actually, only the lightest foamed concrete types (with a density up to 600 kg/m³) are sufficiently lighter than peat to enable the construction of low-settlement pavement structures in areas with weak soil conditions. The Dutch test pavement structures (subsection 4.5.1) are illustrative examples where an EPS layer was built-in under the foamed concrete of 650 kg/m³ density to enable an adequate pavement weight reduction.

The major problem regarding the application of foamed concrete as a light-weight sub-base material is its long-term water absorption. An excessive absorption of water, greater than anticipated in the design, can minimize the advantages of using foamed concrete even in the case of above-mentioned lightest types.

The following subsections contain a description of the basic ingredients and the production process of foamed concrete. Furthermore, material properties are presented from the literature, with emphasis on the aspects of importance for sub-base materials. In subsection 4.5 characteristic Dutch pavement structures and test sections with a sub-base of foamed concrete are presented. Based on data given in the literature and a limited number of linear-elastic multi-layer analyses, representative sub-base stress conditions are defined for the foamed concrete layer.

Subsection 4.6 deals with our materials research on a light foamed concrete, formulated to acquire an insight into the behaviour of this material in previously defined conditions. In subsection 4.7 the conclusions drawn from the literature study and the performed materials research are given.
4.2 FOAMED CONCRETE CONSTITUENTS

The basic ingredients for production of a foamed concrete mix are cement, sand of 2 mm maximum size and water with a preformed foam. Instead of natural sand fly ash can also be used. Besides fly ash either slag, crushed dust, polystyrene granules or lytag powders can be added as foamed concrete fine filler. In order to maintain sufficient workability at low water/cement ratios (w/c) admixtures can be used while accelerators can be applied to reduce the time of setting and/or to obtain a sufficient 'green' strength.

As already mentioned in the introduction of this chapter, only the foamed concrete types with a density lower or equal to 600 kg/m³ can be applied as a light-weight sub-base material. The densities of the basic mix ingredients are as follows: portland cement - 3150 kg/m³, loose sand - 1600 kg/m³, the foam - between 25 and 80 kg/m³ [6]. Figure 4.1 graphically presents the resulting weight of foamed concrete mixes per cubic meter by taking the densities of the constituting materials into account. Furthermore it is apparent that the light foamed concrete mixes used in pavement structures may contain only a limited amount of sand because the maximum allowable density of a light-weight sub-base material is almost reached by mixing cement and water with a preformed foam. Due to evaporation of the (chemically and physically) unbound water the density of the hardened foamed concrete is between 50 and 70 kg/m³ less than the density of the flowing foamed concrete mix [4].

The minimum density of a foamed concrete (approximately 400 kg/m³) is conditioned by a minimum cement content needed for a stable slurry, on one hand, and by achieving some bearing capacity of the foamed concrete after hardening, on the other.

Generally, there are two essential elements in the foamed concrete, namely binding materials in the form of the cement paste matrix and air cells. The air cells generally have
Foamed Concrete as Light-Weight Sub-base Material

a diameter between 0.2 and 0.5 mm. The micro-cellular structure of foamed concrete is illustrated in Figure 4.2. In practice, not only air cells but also small holes with a diameter of a few millimeters (incidentally even up to 10 mm) are occasionally present inside the hardened foamed concrete. The total air content amounts between 30 and 80% by volume [2] depending on the density of foamed concrete mixes.

4.3 PRODUCTION PROCESS

Foamed concrete is produced by mixing a cement slurry or cement paste (with or without additional ingredients) with a considerable quantity of preformed foam by which a large number of air-bubbles are created in the mix. Nowadays, the most often applied production method is foam injection in the pump pipe. Following this method the cement slurry and the foam are separately made and mixed in the very last phase of the production process as shown in Figure 4.3.

The cement slurry or cement paste can be made either directly on construction site or centrally on a concrete plant. Logistical reasons are predominant in making the choice between these two alternatives. The production on a concrete plant results in higher transportation costs because the slurry is very plastic and the truckmixers cannot be fully filled. However, if produced on site the dry ingredients can be (separately or partly mixed) transported and stocked in silo’s near the worksite.

The foam is formed by a foam generator from a foam concentrate, water and compressed air in pre-determined quantities. The foam concentrate is usually diluted in the ratio of one part of concentrate to between five and forty parts of water. The foam is produced by forcing the described mix through a restriction.

Mixing of pre-formed foam and cement slurry takes place in the mixing tube just before pouring of the free flowing foamed concrete mix starts. The mix density can be reduced by increasing the foam content. The foamed concrete does not need compaction.
4.4. FOAMED CONCRETE PROPERTIES

4.4.1 Mechanical properties of foamed concrete

The foamed concrete properties such as strength, stiffness, water absorption and durability are related to its constituents, i.e. water/cement ratio (w/c) and filler and foam contents. Simultaneously, these constituents are determining the mix density, pointing out the correlation between the foamed concrete material properties and its density. The relationship between the mechanical properties and density is not linear.

Being a cementitious material foamed concrete is usually tested according to the methods prescribed for ordinary concrete. However, extra attention is needed regarding mechanical test set-ups with a small contact area between the specimens and supports, e.g. the set-up for four-point-bending test or splitting tensile strength test. In such cases the occurring concentrated contact stresses could cause local surface damage of the foamed concrete specimens (affecting the resulting deformations) and that has to be prevented by implementation of appropriate contact plates which distribute the load. Description of the test procedures and specimen dimensions (including details about the ingredients and manufacturing of the foamed concrete) are given in the literature [7].

Cubic compressive strength, tensile (flexural) strength, elasticity modulus and creep factor which were found in literature for light-weight foamed concrete mixes will be presented in the following paragraphs.

4.4.1.1 Strength and density relationship

According to the literature [4, 8] the compressive strength of foamed concrete seems to be dependent to a larger extent on the density of the mix, i.e. the percentage of air inside, than on the water/cement ratio (w/c). Furthermore, the factors which affect the foamed concrete strength are: cement type, grain shape and size of the sand, presence or absence of fine fillers, and diameters and distribution of air cells [8]. In fact an increasing quantity of cement contributes to the strength up to a certain optimum water/cement ratio. Beyond that optimum ratio further addition of cement has a limited effect regarding the mix strength. However, light-weight foamed concretes can be composed by using a below-the-optimum cement quantity. For instance, compression strength values of 1.55 [9] and 2.0 MPa [8] are reported for foam concrete mixes with an identical density of 600 kg/m³ prepared with 180 and 400 kg/m³ portland cement respectively.

The compressive strength of foamed concrete specimens is determined by compression of 28 days old cubes with 150 mm side length [7]. For densities ranging between 400 and 600 kg/m³ the strength values under compression load, reported in [8, 9, 10, 11], are reviewed in Table 4.1.
Foamed Concrete as Light-Weight Sub-base Material

<table>
<thead>
<tr>
<th>mechanical property</th>
<th>age (day)</th>
<th>Foamed concrete mix</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>fc1(^1)</td>
<td>fc2(^3)</td>
</tr>
<tr>
<td>fresh-mix density (kg/m(^3))</td>
<td>0</td>
<td>400</td>
</tr>
<tr>
<td>cement (kg/m(^3))</td>
<td>240</td>
<td>200</td>
</tr>
<tr>
<td>compressive strength (MPa)</td>
<td>7</td>
<td>0.52</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>0.5</td>
</tr>
<tr>
<td>≤100</td>
<td>1.3</td>
<td>1.50</td>
</tr>
<tr>
<td>flexural strength (MPa)</td>
<td>28</td>
<td>0.10</td>
</tr>
<tr>
<td>tensile strength (MPa)</td>
<td>28</td>
<td>0.05</td>
</tr>
<tr>
<td>modulus in compression</td>
<td>28</td>
<td>300</td>
</tr>
<tr>
<td>modulus in bending (MPa)</td>
<td>28</td>
<td>-</td>
</tr>
</tbody>
</table>


**Table 4.1 -** Average values of the mechanical properties of light-weight foamed concrete mixes

Similar to many other building materials it makes a difference whether the tensile strength is determined in a three- or in a four-point-bending-test (flexural strength), or in a direct tensile (tensile strength) or indirect tensile test (indirect tensile strength). The flexural strength of the foamed concrete measured by means of a four-point-bending-test is between 10 and 20% of the compressive strength [7], while the tensile strength of foamed concrete ranges between 10 and 15% of the compressive strength (reference [8]). Alternatively, as noticed in reference [7], the flexural strength is measured to be between 8 and 17% of the compressive strength.

With respect to the ratio between flexural strength and direct tensile strength, foamed concrete mixes do not seem to differ very much from gravel concrete mixes where the flexural strength is approximately two times higher than the direct tensile strength and 1.5 times higher than the indirect tensile strength [12].

### 4.4.1.2 Modulus of elasticity and strain capacity

The response of a foamed concrete specimen to a compressive stress up to 1/3 of the ultimate value (strength), so-called working stress, is a strain which is approximately proportional to the applied stress. The way to determine the ratio between the working stress and the accompanying strain, i.e. the elastic modulus, is prescribed in the Dutch standard NEN 3838 [13]. The reported E-values in compression for light-weight foamed
concrete mixes vary between 300 MPa ($\rho = 400 \text{ kg/m}^3$) and 1,260 MPa ($\rho = 600 \text{ kg/m}^3$), as listed in Table 4.1. The strain capacity under uniaxial compression, resulting from the strength and the modulus data in Table 4.1, would be of the order of 2%. In contrast with EPS (see section 5.4) there will be total failure of this material beyond this strain level.

The elasticity modulus derived from tension stress and strain values, resulting from bending tests varies between 95% ($\rho = 400 \text{ kg/m}^3$) and 55% ($\rho = 550 \text{ kg/m}^3$) of the modulus in compression. This difference is bigger when more cement is used.

The somewhat lower E-value in bending of this material must be explained by the enormous porosity of foamed concrete.

### 4.4.1.3 Creep factor

The creep factor $\gamma_f$, i.e. the ratio between immediate strain value after loading and additional strain due to long-term creep, of foamed concrete subjected to compressive stress is equal to approximately 3.7 [9]. According to [3] $\gamma_f$ of foamed concrete can be between 1.5 and 8.5 times greater than that of gravel concrete and it increases with decreasing strength and increasing air-cells content. The creep is calculated as the total time-dependent strain minus shrinkage.

### 4.4.2 Physical properties of foamed concrete

Physical properties relevant for the use of the foamed concrete in the sub-base layer are its shrinkage, thermal expansion, thermal conductivity and water absorption. These properties have to be studied because of potential damaging effects of temperature gradients and moisture content, and the long-term effect of water absorption on the density of the foamed concrete layer.

#### 4.4.2.1 Shrinkage due to loss of moisture

Foamed concrete, like all other cement-bound materials, shrinks as a result of a drop in humidity. During the mix hardening process, about 20% of the water in a foamed concrete mix becomes chemically and a certain quantity physically bounded. A part of the remaining free water evaporates (drying) causing a volumetric deformation, i.e. shrinkage. The shrinkage rate is governed by the rate of moisture transfer from the interior to the surface. At normal temperatures this process continues in foamed concrete for months rather than days [9]. If the shrinkage rate is high it can lead to the development of cracks [14] (hindered shrinkage).

According to references [8] and [9] the maximum reported shrinkage value is equal to 6.5% (see Table 4.2). Simultaneously, a reduced in-situ shrinkage lower or equal to 1.5% is suggested for practical purposes [8]. The laboratory values are too high most likely because of the small dimensions of specimens and the dry laboratory environment.
Foamed Concrete as Light-Weight Sub-base Material

compared to real structures and sub-base conditions. In comparison with gravel concrete foamed concrete has approximately 10 times greater shrinkage because there is hardly any non-shrinking aggregate but, instead of it, easily compressible air-cells. The lower the density (and the higher the content of air-cells), the greater the shrinkage that is likely to occur in a foamed concrete mix.

<table>
<thead>
<tr>
<th>Physical property</th>
<th>Foamed concrete mix</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh-mix density</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Cement</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Linear shrinkage</td>
<td>%</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>%</td>
</tr>
<tr>
<td>λ - thermal</td>
<td>W/mK</td>
</tr>
<tr>
<td>Conductivity</td>
<td>W/mK</td>
</tr>
<tr>
<td>μ - moisture diffusion</td>
<td></td>
</tr>
<tr>
<td>Resistance number</td>
<td></td>
</tr>
<tr>
<td>Increase in mass</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Due to 150-days</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Water penetration</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Estimated water</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Absorption for a</td>
<td>kg/m³</td>
</tr>
<tr>
<td>10-years period</td>
<td>kg/m³</td>
</tr>
</tbody>
</table>

| Reference | | |
|-----------|---|
| 1 | reference [8] |
| 2 | reference [10] |
| 3 | reference [9] |

Table 4.2 - Average values of the physical properties of light-weight foamed concrete mixes

4.4.2.2 Thermal expansion and conductivity

The coefficient of thermal expansion of foamed concrete, α, ranges between \(6 \times 10^{-6}/K\) and \(8 \times 10^{-6}/K\) [8]. The α-value is, to a certain extent, affected by density (foamed concrete types with a higher density have a higher coefficient α) and moisture degree. Due to a temperature gradient the thermal expansion in a sub-base of foamed concrete can lead to curling of the layer but neither the expected curvature nor the resulting stresses (in the order of a few kPa) seem significant under Dutch climatic conditions.

The coefficient of thermal conductivity, λ, is dependent on density and moisture degree. Dry foamed concrete is a twice as good insulator as the same mix in saturated state. The λ-values reported in literature are listed in Table 4.2. Its good insulation characteristics will affect the temperature distribution through a pavement structure if a foamed con-
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crete sub-base is applied. The effect of freezing on long-term durability of a wet foamed concrete sub-base is not known. These effects are however much lower than in the case of an EPS sub-base because the coefficient of thermal conductivity of EPS is five times lower that the λ-value of foamed concrete.

4.4.2.3 Water absorption

If the water absorption of foamed concrete would be high its applicability as a light-weight sub-base material would be negatively affected to a great extent. A partly filling of the air-cells with water, e.g. when a foamed concrete sub-base lays below the groundwater level, would result in a significant increase of the original mix density. Such an increase of the initial pavement weight due to water absorption in the foamed concrete layer should not occur during the whole pavement design life, usually a 10 to 20 years period. Therefore, it is necessary to assess accurately the long-term maximum water absorption of light-weight foamed concrete types.

The three transport mechanisms contributing to water absorption are capillary soaking, permeability and diffusion. Capillary soaking seems not to be very important since the water penetration depth due to capillarity is reported to be about 20 mm after 6 months while the mass increase was between 1 and 4 kg/m² [10]. Furthermore, the lighter foamed concrete types with relatively wider air voids absorb less capillary water than denser types because the bigger pore diameters are unfavorable for capillary soaking.

Permeability is associated by the imposed pressure and usually by the material being in saturated state. Water is forced through foamed concrete in response to the imposed hydraulic pressure. Consequently, the quantity of water which flows through the foamed concrete sub-base will be (linearly) dependent on the hydraulic pressure of the groundwater at the top and the bottom of the sub-base layer. In general, a high permeability is associated with high porosity [15]. However, the closed cell structure inside the foamed concrete limits its permeability in spite of the high air-cells content.

Diffusion is the mechanism of transport of moisture caused by a difference (gradient) of internal relative humidity. When the internal relative humidity reaches about 50%, water condenses inside the pores. The condensed water zones extend with rising relative humidity changing slowly the initial unsaturated stage into the saturated stage, causing progress of the moisture front. The rate of diffusion is characterized by the moisture diffusion coefficient \( D [m^2/s] \). Moisture transport caused by diffusion is slower inside a material than through the air \( (D_{air} = 25 \times 10^{6} [m^2/s]) \). The ratio between the moisture diffusion coefficient of air and the one of the considered material is defined as the moisture diffusion resistance number \( \mu [-] \). The lower the \( \mu \)-number of a material is, the higher the rate of diffusion that can be expected in such a material. The low \( \mu \)-values valid for light foamed concrete types (ten times lower than for gravel concrete) are listed in Table 4.2

Penetration of moisture in foamed concrete combines features of both permeability and diffusion. At the beginning of the water absorption process, e.g. after immersion, the effects of a humidity gradient (diffusion) are generally overwhelmed by the imposed hydraulic pressure (permeability). That is manifested by a high initial absorption rate.
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Further penetration of liquid water into foamed concrete becomes slower and slower with increasing penetration depth. Hence, the absorption rate delays in time but the moisture degree still continues to rise. The major driving force behind further absorption becomes the moisture concentration difference in the areas behind and in front of the moisture front. In principal, in case a foamed concrete layer is in contact with water on both sides then theoretically the diffusion process continues till the moment the moisture fronts reach each other.

Immersion of 100 × 100 × 200 mm prisms's under water, previously dried at a temperature of 70°C, resulted in an increase of the mix densities from 505 and 630 kg/m³ (after hardening and before drying) to 735 and 810 kg/m³ respectively [16]. Such density changes, occurring after a four-months period, correspond to water absorption levels of 23% and 18% by volume respectively. A similar degree of water absorption, approximately 20% v/v after a 3-months period, was measured in immersed cubes with a 100 mm side length of foamed concrete with 600 kg/m³ density [17]. These high absorption degrees will not occur in a pavement layer in the same time-period because the ratio surface/volume in case of a sub-base is many times lower than in case of laboratory specimens.

Much lower moisture degrees of water absorption (few percents by volume) occurred during one-side exposure to water under different hydraulic pressures [10]. Actually, the water absorption was expressed as mass of absorbed water per surface unity of foamed concrete (kg/m²). Measurement results after a 1/2-year period are presented in Table 4.2. The test conditions intended to simulate the situation of a layer exposed to (ground)water under hydraulic pressures of 0.5, 1 and 2 m at the undersurface. The water absorption on site should be calculated by multiplication of the experimentally found mass absorption per square meter (for a corresponding hydraulic pressure) with the real wet surface.

Within the same research project [10] the absorption due to rain water and capillarity was measured as well. The found values ranged between 1 and 2 kg/m² after a 1/2-year period for foamed concrete mixes with a density between 485 and 550 kg/m³.

In order to make long-term prediction of maximum water absorption possible, the results given in [10] measured during a 1/2-year period are extrapolated. The obtained values are presented in Table 4.2. This extrapolation is a strong simplification of reality and can lead to misinterpretation. The measurements on site [17] resulted in a higher water penetration after 2.5 years than predicted in [10]. Therefore predictions of long-term water absorption, e.g. for the usual pavement design life of 20 years, have to be made in a conservative way (by implementation of a safety factor) in order to be sufficiently reliable.

Permeability is very susceptible to variations in experimental conditions. The situation on site is more complex and processes like crack development can occur which lead to a substantial extra absorption of water by foamed concrete. Early development of cracks can occur because of shrinkage and because of loading the pavement in which foamed concrete is applied, 3 or 4 days after casting instead of after 4 weeks, when material stiffness is not yet fully reached. The slightest crack provides a low-resistant bypass for the permeating water. Furthermore, light-weight sub-base materials are often applied in areas with peat as subsoil. Consequently, the groundwater which surrounds a foamed
concrete layer is likely to be acid with a degree of acidity pH = 5. Experiments [16] with acid water containing citric acid in such proportion that the pH-value amounted to 4 (ten times higher concentration than in reality) resulted in a somewhat higher water absorption rate. Simultaneously, acid water corrodes (to a certain extent) foamed concrete which can lead to additional water absorption.

Summarizing, it can be stated that the literature does not provide very reliable data for reliable prediction of the maximum water absorption in foamed concrete under the sub-base conditions during the pavement design life (usually a 20-years period). By taking into account the long-term (negative) effects of cracks developing due to shrinkage and thermal action and an aggressive environment (acid groundwater), the maximum water absorption of the mixes with densities of 400 and 600 kg/m³ should be estimated to amount 15% and 10% by volume respectively. In other words, as saturated foamed concrete density values of 550 and 700 kg/m³ should be used instead of the initial densities of 400 and 600 kg/m³ for the weight-balance calculations, especially when the foamed concrete sub-base is laid completely below the groundwater table. If a foamed concrete layer lays above the groundwater level a 1.5 times lower maximum water absorption could be expected, i.e. 10% and 7% by volume respectively. One has to keep in mind that an eventual repair of the deep laying sub-base means a complete road reconstruction so that a high safety factor has to be implemented in the design of the lower pavement layers.

4.5 PAVEMENT STRUCTURES WITH FOAMED CONCRETE SUB-BASE

Foam concrete is used as sub-base layer of pavement structures. The pavement design is dependent on expected traffic loading, soil profile and its bearing capacity, available local materials and their prices, the underground infrastructure like cables and piping and their accessibility, available construction time etc. In the next paragraph some Dutch full-scale test sections with a foamed concrete sub-base and a toplayer of either concrete or small concrete blocks will be discussed which have been constructed over a very poor peat subsoil.

4.5.1 Dutch test pavement structures

Full-scale test sections were laid in an area with a thick peat layer as subsoil [18]. The subsoil bearing capacity was so poor that, although the implemented light-weight foamed concrete reduced the pavement weight to a certain extent, it was necessary to apply an EPS layer below it to reach sufficient weight reduction. This fact points out the limitation of foamed concrete with respect to its applicability on very compressible subsoils. The materials and thicknesses of the test sections with foamed concrete are illustrated in Figure 4.4 [18, 19].
This particular research project was carried out to determine which pavement structure(s) could be appropriate for farm to market roads in peat areas. Characteristics of such roads are a narrow cross section because of low traffic intensities. Traffic mainly consists of trucks loaded by agricultural products to and from the farms. Either concrete, concrete blocks or precast concrete slabs were applied as toplayer in the test pavements.

For the top layer concrete type B37.5 (characteristic 28 days cube compressive strength is equal to 37.5 MPa) was found to be suitable for the expected axle loadings. The precast concrete slabs had dimensions of $1.5 \times 2 \times 0.14$ m with steel edges. Foamed concrete had a density of 650 kg/m$^3$ with the exception of the concrete block pavement structure (test section 3) with two foamed concrete sublayers with densities of 650 kg/m$^3$ (upper 0.25 m) and 750 kg/m$^3$ (lower 0.15 m). The built-in EPS foam had a density of 20 kg/m$^3$ (EPS20).

A visual inspection performed after a few thousands standard axle load repetitions pointed out the development of cracks on the sections 1 and 2 with concrete toplayers of 80 and 100 mm respectively. The joints in the concrete toplayer did not behave satisfactorily, cracks were developing between the joints. Those joints, made every 3 m with a depth of 1/3 of the toplayer thickness, were probably not deep enough to weaken sufficiently the structure. On test sections 3 and 4, with concrete blocks and precast concrete slabs respectively, only a small amount of unevenness was observed.

The analysis of falling-weight deflection measurements suggested the existence of a good bond between the concrete and foamed concrete layer on the test sections 1 and 2. General conclusion for all test sections regarding the back-calculated elasticity modulus of the foamed concrete layers was that this material had a minimum modulus as prescribed, 1.000 MPa corresponding to its $E = 650$ kg/m$^3$, if no cracks developed. In case of a cracked concrete toplayer the E-values determined for the foamed concrete were much lower, indicating possible damage of the sub-base.
4.5.2 Representative conditions in foamed concrete layer

Design criteria for foamed concrete layers are the vertical stress at the top and the horizontal tensile stress at the bottom, which occur due to an above passing wheel load. In fact, the vertical and horizontal stresses occurring in a foamed concrete layer should be used for determination of the maximum number of load repetitions before cracking develops. For these purposes fatigue lines belonging to different foamed concrete mixes under both compression and tension are necessary. Unfortunately, no results of cyclic loading tests on foamed concrete have been published in the literature from which proper fatigue lines for the light-weight mixes could be determined.

To determine representative stress values and loading frequencies occurring in a foamed concrete layer, we have analyzed a thin pavement structure with such a layer by means of the linear-elastic multi-layer program BISAR. The pavement structure that was selected for the analysis, was test section 3 (see subsection 4.5.1) with a toplayer of concrete blocks (Figure 4.5). Two different E-values were assumed for the foamed concrete layer, firstly 500 MPa (modelling foamed concrete with a density of 500 kg/m³) and, secondly, 1000 MPa (ρ = 600 kg/m³).

The vertical stress values due to a 50 kN wheel load at the top of the foamed concrete layer were calculated to be 287 kPa (E = 500 MPa, i.e. ρ = 500 kg/m³) and 315 kPa (E = 1000 MPa, i.e. ρ = 600 kg/m³) respectively. The horizontal tensile stress at the bottom of the layer had values of 226 kPa and 284 kPa respectively. It appears from these figures that for both materials the vertical compressive stresses would not be close to the material strength limits. However, the tensile flexural stresses are so high that cracks are likely to develop, especially in the foamed concrete mix with a density of 500 kg/m³.

The loading frequency for the vertical stress at the top of the foamed concrete layer is approximately 16 Hz representing a wheel load passing with a speed of 60 km/h. The corresponding loading frequency for the horizontal tensile flexural stress at the bottom of the sub-base layer is equal to 8 Hz.

4.6 MECHANICAL TESTING OF FOAMED CONCRETE

Mechanical properties of light-weight foamed concrete types as reported in literature (see Table 4.1) show a large degree of variation. The variations in reported strength and modulus values of mixes with identical densities are not only caused by different ratios between the mix constituents but also by the quality of the mix constituents, the degree
of water absorption, the test specimen preparation procedure and test conditions. In order to get a more accurate characterization of the foamed concrete material properties we have set up and carried out a lime material testing program.

In this testing program research was performed on a selected light-weight foamed concrete mix with a density, as reported by the producer, of 500 kg/m$^3$ (see Appendix I). It should be noted that this is the slurry density. The mix is representative for the foamed concrete types used to a considerable extent as a light-weight sub-base material in (Dutch) pavement structures. Full details about the mix constituents are given in Appendix I.

Foamed concrete cylinders, 300 mm high with a diameter of 150 mm, were subjected, firstly, to monotonic compression tests. Secondly, cyclic compression loading tests were performed, again on cylindrical specimens with above-mentioned dimensions. Thirdly, indirect tensile tests were carried out on 50 mm thick foamed concrete specimens previously cut off from the cylinders. Finally, small foam concrete beams (h x w x l = 50 x 50 x 300 mm) were subjected to three-point-bending tests.

### 4.6.1 Compression tests

In order to investigate the general stress-strain behaviour of the considered light-weight foamed concrete type under representative sub-base stress conditions, strain-controlled compression tests were performed on the cylinders of this material. The maximum strain to which the specimens were compressed in the test series was equal to about 0.1%, corresponding to a compressive stress of about 300 kPa.

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**Figure 4.6** - Compression test set-up including the unit for computerized data acquisition

**Fig. 4.7** - Compression test set-up with the especially designed vertical displacement transducer set up
A picture of the test set up with a foamed concrete specimen is given in Figure 4.6. Figure 4.7 shows one of two symmetrically placed displacement transducers; both transducers are fixed on the loading plates. As measurement result the average of the vertical deformations measured by both transducers was taken.

It is apparent that the elasticity modulus of the considered foamed concrete type is significantly lower than the corresponding values in Table 4.1 (about 300 MPa instead of 650 MPa). Figure 4.8 is illustrative for foamed concrete response under compression.

4.6.2 Uniaxial cyclic loading tests

Uniaxial cyclic loading tests on the particular light-weight foamed concrete were carried out in two series each with a different cyclic stress level $\sigma_c$. The cyclic stress components were chosen as realistic stress values occurring in the foamed concrete layer due to a 100 kN standard axle-load in both a common and a thin pavement structure. In the first test series, $\sigma_c$ amounted about 140 kPa and in the second one 250 kPa. Specimens were subjected to 100,000 load repetitions. Such a relatively high number of loading cycles was assumed to be sufficient for the development, if any, of permanent deformation or cracks.

The loading frequency was dictated by equipment limits. The higher the cyclic stress levels, the lower the frequency of the stress-controlled pulses (with a haversine shape) that could be applied. In the case of $\sigma_c = 140$ kPa a loading frequency of 5 Hz was feasible while for $\sigma_c = 250$ kPa the limit frequency was 4 Hz.

The static stress component, $\sigma_s$, was equal to about 10 kPa simulating the dead weight of the pavement layers above the foamed concrete sub-base. At the beginning and at the end of the test a static load was maintained for some time.

Total applied stresses (total $\sigma = \sigma_s + \sigma_c$) and measured corresponding strain (total $\epsilon = \epsilon_s + \epsilon_c$) values for one of the three tested specimens are graphically illustrated in Figure 4.9. During the tests the deformations remained very limited while a small permanent deformation developed.
Figure 4.9 - Total applied stress and measured strain values in the first uniaxial cyclic loading test series on the light-weight foamed concrete with a density of 500 kg/m³

The dynamic moduli $E_{dyn}$ of the foamed concrete calculated from the dynamic cyclic stresses and strains, are shown in Figure 4.10. From this figure it is evident that the dynamic strain component did not increase under the applied cyclic stress. $E_{dyn}$ ranged between 290 and 320 MPa being significantly lower than the values reported in the literature for the foamed concrete mixes with a density of 500 kg/m³ (see Table 4.1). The explanation could be that the reported E-values were determined under higher stress levels when the material stiffening developed. The beginning of such a material stiffening is illustrated in Figure 4.8 where actually less than one fifth of the complete stress-strain curve (up to the break point) is presented.

Figure 4.10

Dynamic modulus of elasticity of the foamed concrete with density of 500 kg/m³ resulting from the cyclic stress and strain components in the first test series

Figure 4.11 shows the total applied stress of about 250 kPa and the resulting strain values found on a foamed concrete cylinder in the second test series. Comparing the permanent strain values measured after 100,000 load repetitions to those in the previous test series, the permanent strain has relatively higher but still very small values.
Figure 4.11 - Total applied stress and measured strain values in the second maximum cyclic loading test series on the light-weight foamed concrete with density of 500 kg/m³.

The values of the dynamic modulus $E_{dy}$ are illustrated in Figure 4.12. The figure shows that the $E_{dy}$ had constant values and that the dynamic strain did not increase under the performed cyclic loading. In other words, similar to the previous test series, the dynamic behaviour of the analyzed light-weight foamed concrete type was elastic. In this case however a higher dynamic modulus $E_{dy}$ was found (compare Figures 4.10 and 4.12). It seems to correspond with the stress-strain material behaviour in compression which is illustrated in Figure 4.8. The material modulus increases with increasing stress and strain values (at least if $\sigma$ and $\epsilon$ stay within the considered stress-strain region which is representative for the values expected in the foamed concrete sub-base) but the effect may also be partly due to surface irregularities. The dynamic modulus $E_{dy}$ under cyclic loading of 250 kPa ranged between 400 MPa and 500 MPa for the tested specimens. Although these $E_{dy}$-values are higher than those in the case of $\sigma_c = 140$ kPa, these values are still lower than the $E_{dy}$-values reported in the literature, and listed in Table 4.1.

Figure 4.12

Dynamic modulus of elasticity of the light-weight foamed concrete with density of 500 kg/m³ resulting from the cyclic stress and strain components in the second test series.
4.6.3 Indirect tensile tests

Both compression and uniaxial cyclic loading tests provided information about the stress-strain response of the considered foamed concrete mix under compressive stresses. In subsection 4.5.2 it has already been pointed out that the tensile stresses occurring at the bottom of the sub-base layer seem to be the dominant design criterion. Tensile strength of foamed concrete amounts to only 15% of its compressive strength and this value is more probable to be exceeded than the compressive strength at the top.

Performing direct tensile tests was not considered to be suitable since gluing the specimens to steel loading plates would be difficult because of the foamed concrete porosity. Therefore indirect tensile tests were carried out on foamed concrete specimens instead of direct tensile tests.

The principle of the indirect tensile test is simple: through compression tensile stresses perpendicular to the loading surface are created in the specimen. Figure 4.13 gives an impression of the way the crack develops if loading continues to increase (a and b) and the stress distribution in the cross sections.

![Diagram showing stress distribution](image)

Figure 4.13
Loading position and crack development as well as stress distribution in the disk cross section during an indirect tensile test

On the basis of linear elasticity the horizontal tensile stress, \( \sigma_h \), in the centre of the specimen can be calculated:

\[
\sigma_h = \frac{2F}{\pi l d}
\]  

[eq. 4.1]

where:  
\( \sigma_h \) - horizontal tensile stress [Pa]  
F - vertical force [N]  
l - length of the specimen [m]  
d - diameter of the specimen [m]
Equation [eq. 4.1] is only valid till the moment a crack occurs because then the material does not behave elastically anymore. For many materials the indirect tensile test is used for assessment of the strength characteristics under monotonic loading as well as for the development of fatigue relations (Figure 4.14). On the basis of such a fatigue curve the maximum number of standard axle load repetitions, $N$, that a pavement structure can sustain until cracks develop can be determined knowing the maximum horizontal stress value in the layer.

A problem associated with the indirect tensile test on foamed concrete specimens is the contact stress under the loading plates. The contact stress can exceed the compressive strength before the tensile stress causes cracks in the specimen. To avoid high contact stress values, a wider loading strip was used (1 inch instead of $\frac{1}{2}$ inch) than the one normally used on other materials. Furthermore, the curvature of the loading strip was the same as the specimen, all these to avoid stress concentration on a specimen. Indirect tensile test set-up including a foamed concrete specimen is illustrated in Figure 4.15.

Results of a displacement controlled monotonic indirect tensile test are shown in Figure 4.16. During the strain-controlled loading (the displacement rate was about 0.7 mm/s) a discontinuity was obtained in the force gradient. A short-term decrease and a further slower increase of the force value (Figure 4.16) were caused by a local failure of foamed concrete which occurred directly under the loading strip due to the stress concentrations.

According to Figure 4.16 and the results of other performed indirect tensile tests it was expected that the critical stress concentration should not occur for a force value smaller than 1,000 N.
In an attempt to investigate whether cyclic loading with this force could result in crack development in the foamed concrete specimen (prior to local failure due to the stress concentration) cyclic tensile tests were carried out. A low and high load level were used in the cyclic tests (500 N resp. 1000 N, see Figure 4.17) and the resulting vertical displacements were recorded. The loading frequency was equal to 8 Hz, the number of load repetitions was equal to 100,000.

Even though a certain permanent vertical deformation was obtained due to the cyclic loading there was no indication of cracks developing in the middle of the specimen caused by tensile stresses. The measured deformations took place under the loading plate, and were the result of compressive stresses. Consequently, cyclic indirect tensile testing on foamed concrete appears not to be an adequate approach to provide experimental data necessary for establishing the fatigue curve of the analyzed mix.
4.6.4 Three-point-bending tests

As an alternative for the indirect tensile test, three-point-bending tests were carried out which resulted not only in sufficiently high horizontal tensile flexural stress values but, simultaneously, led to lower contact stresses than in the prior tests. The foamed concrete specimens were beams with dimensions $h \times w \times l \ 50 \times 50 \times 300$ mm cut from the cylinders and laid on supports with a distance of 250 mm between them. The used three-point-bending test set-up is shown in Figure 4.17. The horizontal tensile stress is calculated using:

$$\sigma_h = \frac{3 FL}{2wh^2} \quad \text{[eq. 4.2]}$$

where: $\sigma_h$ - horizontal tensile flexural stress [kPa]
$F$ - vertical force [N]
$L$ - distance between supports [mm]
$h$ - height of the beam [mm]
$w$ - width of the beam [mm]

First of all foamed concrete beams were tested until failure in a displacement-controlled test in order to determine the maximum force values to which the beams could be loaded. Then dynamic three-point-bending load-controlled tests were performed with force values between 40% and 70% of the previously measured flexural strength.

The results of a displacement-controlled three-point-bending test are graphically illustrated in Figure 4.18. The beam collapsed at a force level of approximately 80 N. Corresponding horizontal tensile stress on the beam’s underside in the loaded cross-section was equal to about 230 kPa ([eq. 4.2]), which is somewhat higher than the highest reported flexural strength values (Table 4.1).
Foamed Concrete as Light-Weight Sub-base Material

Figure 4.18
Applied vertical force and vertical displacement of the loading piston during three-point-bending test on foamed concrete

Dynamic three-point-bending tests were performed in the load-controlled mode at a frequency of 8 Hz. The lower value of the cyclic force was equal to 30 N and was dictated by the minimum force the equipment could generate while the maximum applied force had a value of 50 N. Under higher cyclic forces failure was obtained almost immediately. An example of the test results is graphically illustrated in Figure 4.19.

Figure 4.19
Applied vertical force and number of load repetitions until failure during dynamic three-point-bending test on foamed concrete

Exposure of the foamed concrete beams to cyclic loadings of 50 N resulted in failure after 500 to 4300 load repetitions; it should be noted that a range of a factor 10 is normally observed in fatigue testing. At a 40 N force one beam failed after 154,600 loading cycles but in the next test more than $5 \times 10^4$ repetitions were not enough.

In order to assess the tensile flexural strength of the mix more accurately dynamic three-point-bending tests were repeated with increasing force values. Starting force was equal to 40 N and this value was increased by 10 N after each 15,000 load repetitions. Again, a large variation was found in the force levels which caused failure. The wide dispersion of the maximum cyclic flexural tensile stress values which foamed concrete can sustain is probably dependent upon presence or absence of either micro cracks or weak spots at the location of stress concentration. However, the force level of 40 N and its corres-
Foamed Concrete as Light-Weight Sub-base Material

ponding flexural tensile stress of 115 kPa seemed not to be critical in almost all tests. It seems reasonable to assume that the minimum values of the flexural tensile strength reported in the literature and listed in Table 4.1 are realistic.

From the limited test results it can be concluded that there was a slight tendency for wet beams to behave better than dry beams. However the number of tested beams was far too low to draw definite conclusions on this.

4.7 CONCLUDING REMARKS ON FOAMED CONCRETE

- Of all foamed concrete mixes with densities between 400 and 1600 kg/m³ only the lightest foamed concrete types (with a density up to 600 kg/m³) are appropriately lighter than subsoil to enable the construction of low-settlement pavement structures in areas with weak soil conditions. The replacement of the excavated soil by foamed concrete with medium or high density would not allow a sufficient pavement weight reduction.

- The literature does not provide reliable data for accurate prediction of the maximum water absorption in foamed concrete under the sub-base conditions during the pavement design life (usually a 20-years period). By taking into account the long-term (negative) effects of cracks developing due to shrinkage and thermal action and an aggressive environment (acid groundwater), the maximum water absorption of the mixes with densities of 400 and 600 kg/m³ is estimated to be at least 15% and 10% by volume respectively for foamed concrete sub-bases laid below the groundwater level. In the cases of pavement structures with a foamed concrete layer laid above the groundwater level 1.5 times lower maximum water absorption is expected, i.e. 10% and 7% by volume respectively.

- The maximum vertical stress values occurring in the foamed concrete sub-base hardly exceed 20% of the material (28 days) compressive strength. Therefore the design criterion regarding the maximum allowable vertical stress will be satisfied if foamed concrete is dried properly and becomes sufficiently hard before loading.

- Experimentally determined E-values in compression of the foamed concrete with mix density of 500 kg/m³ were lower than the values reported in the literature. Furthermore, for strains up to 0.1% the effective elasticity modulus seems to increase with increasing stress level.

Under the maximum vertical stress values expected in pavement structures, which is less than 20% of the material compressive strength, the experimentally determined moduli were equal to about 2/3 of the reported E-values while under twice lower stress level the experimental E-values did not exceed 1/2 of the corresponding moduli in the literature.
Foamed Concrete as Light-Weight Sub-base Material

- There is a wide dispersion of the maximum cyclic tensile stress levels which foamed concrete can sustain before breaking occurs, probably due to presence or absence of either micro cracks or weak spots at the location of stress concentration. However, for tensile flexural stress values of 115 kPa, or less, there is no need to worry about fatigue.

- It is obvious that more work has to be done to define the fatigue curves for lightweight foamed concrete types under tensile stresses. Not only the effects of different densities but also the question to which extent a varying mix composition and a varying moisture content affect the mechanical behaviour of foamed concrete should be studied. Dynamic three- or four-point-bending tests seem to be an adequate testing approach for such a research program.

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CHAPTER 5

MATERIALS RESEARCH ON EPS

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5.1 INTRODUCTION

Expanded Polystyrene foam (EPS) is a material with a long list of applications which has been growing continuously in the last decades. Since the early seventies, EPS as an extremely good insulator has been built-in in pavement structures in Norway. In these pioneer days the purpose of the EPS layer was to prevent freezing of the frost-susceptible soil. The closed cell structure of EPS results in a very low thermal conductivity and density, its two main material characteristics. Positive experiences with EPS as an insulating layer led to the next step, the use of EPS as a light-weight sub-base material.

The wide use of EPS for thermal insulation in buildings led to the standardization of its mechanical properties. The big difference between the representative loading and conditions valid for thin EPS insulating boards and those valid for EPS blocks in pavement structures presents a problem. EPS, in a sub-base, is exposed to static loads (dead weight of the overlying layers) and to dynamic loads (traffic loads). All this takes place in surroundings where the moisture content and the temperature vary daily and seasonally. Therefore, the existing requirements pertaining to mechanical properties of EPS must be carefully reconsidered and adjusted for the application of EPS in the sub-base of pavement structures.

The determination of the mechanical behaviour of EPS as a sub-base material implies a three-stage procedure, namely assessment of the representative moisture, temperature and loading conditions, the simulation of that particular state and the determination of the corresponding EPS material parameters. The representative loading, moisture and temperature conditions have to be defined by means of either modelling and calculating, or measuring and evaluating as described in Chapter 3.

This chapter deals with the investigation of the mechanical characteristics of EPS with bulk densities of both 15 kg/m³ (EPS15) and 20 kg/m³ (EPS20) under different hygro-thermal conditions. In the experiments the EPS is tested under the most severe expected loading, moisture and temperature conditions that will occur in an EPS sub-base.

Section 5.2 describes the production process of EPS. The material has a closed cell structure in which more than 95% of the volume is occupied by cells. Furthermore, details are given about EPS20 and EPS15 specimens on which our experiments were carried out. The details concern dimensions and relevant process characteristics for each of in total three sets of EPS20 and one set of EPS15 samples delivered in the course of the research.

Section 5.3 deals with our hygro-thermal research on EPS. Water absorption of EPS is simulated and stimulated in different ways. The various subsections in this section discuss the water absorption by means of immersion, immersion together with exposing to underpressure, and exposing to freeze-thaw cycles. The purpose of the experiments was twofold. Firstly, to obtain information on the water absorption of EPS under
different conditions and, secondly, to prepare samples for mechanical research. Subsequently, the influence of water absorption and treatment under extreme temperatures on the mechanical behaviour of EPS15 and EPS20 is determined by using these samples.

Section 5.4 describes compression tests on cylindrical specimens of EPS15 and EPS20. Compression was performed with different strain rates on specimens with varying moisture contents. The EPS specimens were exposed not only to water absorption but to freeze-thaw cycles as well. The linear-elastic region, the EPS material modulus and the stress and strain values corresponding to beginning of permanent deformations were determined. Different strain rates, various test temperatures and specimen conditions were used to determine the effects, if any, of these test conditions on EPS15 and EPS20 material behaviour.

In Section 5.5 the creep in the EPS sub-base is investigated under static stresses which represent the dead weight of the overlaying layers. This creep leads to an increase in the initial deformation in EPS. Consequently, additional settlements of the pavement structure will occur after completion of the construction. In this section the experiments, carried out in an attempt to predict the final level of the additional settlement, are described. EPS15 and EPS20 were loaded at two different static stress levels representing dead weights of thin and thick overlaying layers above an EPS sub-base.

Uniaxial cyclic loading tests are discussed in Section 5.6. In these tests the samples have been exposed to combined static and dynamic loading. The static and dynamic stress components used in the first test series on EPS20 represent the maximum expected stress levels in the sub-base. In the second test series on EPS20 as well as EPS15, the specimens were loaded up to stress levels exceeding the maximum expected values, leading to significant plastic deformations. The purpose was the determination of elastic and permanent deformations of EPS15 and EPS20 caused by a wide range of stress levels. Similarly to the compression tests, the experiments on EPS20 were performed at low temperatures too. The samples were classified in three sets. The first set contained dry samples, the second set the wet samples and the third set represented the samples exposed to freeze-thaw cycles.

In Section 5.7 the dynamic moduli of elasticity of EPS20, measured by electro-dynamic methods, are discussed. The first section describes the determination of $E_{\text{dyn}}$ by means of the propagation velocity of a wave through a prismatic sample. In the second section, $E_{\text{dyn}}$ is determined by means of the fundamental frequency. Finally, the results are reviewed and conclusions are drawn from the values obtained for $E_{\text{dyn}}$ of EPS20. Modulus assessments using ultrasonic techniques have been done in order to determine to what extent these techniques could be used for quality control purposes.

Finally, in Section 5.8 the overall conclusions pertaining to the materials research on EPS15 and EPS20 are drawn. A list of references concludes this chapter.
5.2 EXPANDED POLYSTYRENE FOAM

5.2.1 Production process

Expanded polystyrene foam (EPS) is manufactured from expandable polystyrene beads which are moulded and fused in block-moulds using dry saturated steam. The polystyrene beads themselves are produced by polymerisation of styrene monomer in an aqueous suspension. During the polymerisation, which takes place in a reactor with a stirrer system, an expansion agent is added. This blowing agent, normally pentane, is absorbed by the expandable polystyrene beads and enables their expansion in the later production phase. After completion of the suspension polymerisation, the beads are separated from the water by centrifuging and drying. The dry expandable polystyrene beads are distributed with respect to their size by sieving. The beads are then coated to optimize the later conversion processes such as expansion and moulding.

![Production process of EPS foam diagram]

The expanded polystyrene foam (EPS) is not directly obtained from the expandable polystyrene beads. These first have to be pre-expanded, an operation performed by exposing the beads to dry saturated steam, being the most efficient expansion medium due to its high heat content and diffusion rate through polystyrene. The pre-expansion occurs above both the glass transition temperature of the EPS matrix and the boiling point of the absorbed expansion agent in the beads incorporated during the polymerisation process. In this way, the polystyrene beads expand and form a cellular structure inside the softened polymer, called pre-foam. After maturing, during which air diffuses into the newly-formed cells, this pre-foam is moulded and fused in block-moulds, again under the influence of dry saturated steam. The final product is a low-density EPS foam. The whole manufacturing process, described in detail in the literature [1, 2], is schematically represented in Figure 5.1.
The solid density of polystyrene, equal to 1030 kg/m³, is reduced to the range of 15 to 35 kg/m³, depending on the type of EPS. The high air content of the foam (in jest, EPS is sometimes called 'well-packed air') as well as its closed cell structure result in a very low thermal conductivity and density, two main EPS characteristics.

5.2.2 EPS20 and EPS15 test specimens

Three sets of EPS20 specimens, an EPS20 block and a set of EPS15 specimens were provided by the Shell Chemical Research Centre, Louvain-la-Neuve, Belgium, on three occasions for the materials research described in this chapter. The first EPS20 specimen set contained beams with a cross section of 50 by 50 mm and a length of 300 alternatively 500 mm, which were tested by electro-dynamic methods discussed in Section 5.7. In this test series the above-mentioned EPS20 block was also used. The second EPS20 set contained cylindrical specimens 200 mm high with a 100 mm diameter. These specimens were used for hygro-thermal research on EPS20 (discussed in Section 5.3), compression tests at normal and low temperatures (Section 5.4), and determination of long-term creep behaviour (Section 5.5) as well as for uniaxial cyclic loading tests (Section 5.6). The third EPS20 specimen set and the only EPS15 set included cylinders 300 mm high with a diameter of 150 mm. On these specimens compression tests (Section 5.4), creep tests (Section 5.5) and uniaxial cyclic loading tests (Section 5.6) were carried out.

The test set-up dictated the dimensions of the cylindrical EPS specimens. A small load cell, equipped with both vertical and horizontal transducers and used for cyclic loading tests, is designed for small cylindrical specimens (Ø 100 mm, H = 200 mm). Determination of Poisson’s coefficient was possible only on such relatively small cylinders. However, for the second series of the compression and cyclic loading tests on both EPS15 and EPS20 relatively larger specimens (Ø 150 mm, H = 300 mm) were used. This was done to minimize the edge effects on test results even more than in the first series.

The cylindrical EPS samples used for the materials research were cut out of an EPS block using a hot wire. Relevant details regarding the EPS20 and EPS15 production process including the duration of certain process phases, temperatures and pressures, are listed in Appendix II. The intention was to obtain material samples which do not differ from EPS produced in a factory. Therefore the EPS tested was not subjected to any special treatment during the production process.

5.3 HYGRO - THERMAL RESEARCH

5.3.1 Test specifications

Hygro-thermal research on EPS20 was carried out for two reasons: to obtain information on the water absorption stimulated in different ways, and to prepare EPS samples for further mechanical testing. In the scope of this investigation, EPS20 samples were subjected to repeated freeze-thaw cycles, being kept under water for a long time, and
exposed to underpressure. Different from EPS20, the EPS15 specimens were treated only by immersion and freeze-thaw cycles as preparation for further mechanical testing. The final aim of different treatments was to determine the influence of a (relatively) high level water absorption and/or of a low temperature on the mechanical characteristics of EPS.

In comparison to other light-weight sub-base materials, EPS has a low water absorption. The dominant transport mechanism responsible for faster water absorption seems to be diffusion combined with capillary condensation. EPS is practically an impermeable material for water. This is related to the closed cell structure of the material. Previous measurements by the author [3] indicated no significant absorption of water by means of capillary suction or hygroscopic attraction. A reasonable assumption is that diffusion of vapour occurs in the cell space and through the very thin cell walls because of the existing difference of moisture concentration inside and outside the cells.

EPS contains only a few volume percent of water in the saturated state. Nevertheless, those few volume percent of water absorption represent a very large relative increase of the EPS volume weight. Each volume percent of water absorption means, in the case of EPS20, an increase of 50% of the volume weight $\rho_{\text{EPS}}$ because of the extremely low original (dry) volume weight.

### 5.3.2 Water absorption by immersion

The purpose of immersing EPS, besides measuring the increase of the water absorption level in time, was to prepare samples for mechanical research. The (experimental) problem is that the immersed EPS absorbs water very slowly. This way of water absorption was used as a reference for water absorption accelerated by means of underpressure or exposure to freeze-thaw cycles.

EPS is a material about 50 times lighter than water. Consequently, the immersed EPS samples were loaded by ballast to keep them under water. As shown in Figure 5.2, the ballast was fixed on a frame with dimensions appropriate for the EPS sample. The samples were small cylinders in the case of EPS20, approximately 200 mm high with a 100 mm diameter, and large cylinders, $\varnothing = 150$ mm and $H = 300$ mm, in case of EPS15.

The measurement of water absorption of EPS20 was performed in accordance with the ISO-method 2896 for cellular plastics. The EPS20 samples were weighed by hanging the frames by a wire with a hook on an accurate balance without removing them from the water. Otherwise, it would always be questionable whether the weighed water is located at the sample surface or inside the EPS. The average room temperature was about 18°C.
The total water absorption of EPS20 amounted to 1.54% v/v after a period of one year. This is the average total water absorption from 12 cylindrical samples. The measured values actually varied between 1.12 and 1.77% v/v [4]. The average water absorption versus time is shown in Figure 5.3a. The gradient of the curve decreased significantly after a month and it became practically negligible after 12 months. Accordingly, the increase in water absorption became very small and near the limit of detection. In Figure 5.3b the absorption is shown as a function of time on a log scale. The asymptotic value of 2% v/v can be considered as the maximum percentage of water that EPS20 samples of this size will absorb by immersion.

![Graph showing water absorption of EPS20](image)

**Fig. 5.3a** - Water absorption of EPS20 by means of immersion

![Graph showing water absorption of EPS20](image)

**Fig. 5.3b** - Water absorption of EPS20 by means of immersion (with log scale for time)

The EPS20 samples have been kept for approximately one year in water until the moment they were used for further research. Being immersed over such a long period the samples were considered to be in a representative state.

The experiment resulted in lower levels of water absorption of EPS20 than the values indicated in the literature [5, 6, 7]. The reason should be sought in the form of the tested samples, and in the way the samples were cut from a large EPS block.

In insulation, EPS is applied in the form of ‘thin’ boards for covering walls. The samples tested in the literature were also boards with a much larger surface-to-volume ratio than the cylindrical samples used for the experiments described in this section.

The cutting of samples from a large EPS block, carried out by a hot wire, implies melting of the cells at the sample’s surface, making penetration of water more difficult. Surface cells remain open on samples sawn from a block.

EPS15 specimens were immersed mainly for the purpose of preparing specimens for further mechanical testing and not for accurate determination of absorption level as in the above-mentioned case of EPS20. EPS15 specimens, 32 in total, were cylinders with 150 mm diameter and a height of 300 mm. Given the results shown in Figures 5.3a and 5.3b and considering the time consumption, it was decided to immerse the cylinders
during two months before testing them. This period was twice as long as the period of 1 month after which a substantial decrease in absorption rate of EPS20 had occurred.

The total water absorption of EPS15 amounted to 1.56% v/v, an average taken from 32 cylindrical samples, after a period of two months. The measured values varied between 1.25 and 1.87% v/v. Standard deviation was equal to 0.18 % v/v. The measured water absorption rate and range were higher as compared to those values found for EPS20. This is in accordance with literature, lighter EPS types absorb more water than the heavier ones. However, the measured values were again lower than those mentioned in literature for EPS15 (up to 5 % v/v [6]). The explanation for the difference seems to be identical to those for EPS20, i.e. ratio between the sample's surface and volume was small and the samples had been cut using a hot wire.

### 5.3.3 Stimulating water absorption by underpressure

The immersed EPS absorbs water very slowly. Getting EPS into a wet state requires the EPS samples to be kept under water for a long period. Therefore, preparation of EPS in this way for further materials research is a very time consuming procedure. In an attempt to reduce the time needed for the wetting of EPS samples, efforts were made to stimulate the water absorption of EPS20. The first procedure applied involved the use of vacuum.

Dry EPS20 samples were placed in a vacuum kettle and subjected to vacuum. The air in the cells was pumped out by the creation of a pressure gradient between the inside of the material and the surrounding space. The underpressure draws water between the beads of EPS from the moment that water is passed in the kettle. Vacuum was maintained during a day after immersion of the EPS20. If total vacuum (-101 kPa) is realized inside the cells, water is drawn into the material by a 101 kPa pressure.

Stimulating the water absorption in this way resulted in absorption of a relatively large amount of water by EPS20 in a short period; indeed much faster than due to immersion. However, the EPS20 samples became deformed, as illustrated in Figure 5.4, and became unusable for further tests. The mechanical damage to the samples excluded them from further use.

In an attempt to accelerate the water absorption without negative effects, samples were subjected to a partial vacuum. The approach was based on creating a limited underpressure in the cells which would draw water in the material but, in contrast with vacuum, the pushing pressure behind had a value lower than 101 kPa. In this case, EPS absorbs water more slowly but the risk of deformation of the samples is likely to be smaller.

![Deformation of the EPS samples which are subjected to underpressure](image)
Unfortunately, the damage to the samples was not acceptable, even when the underpressure ranged between -31 and -35 kPa (approximately 30% of vacuum). The deformation of the samples was severe for a created underpressure of around -70 kPa (approx. 70% of vacuum). Therefore, it was decided to stop with stimulating the absorption of water into EPS20 by means of underpressure. It was concluded that this approach was useless with regard to the preparation of wet samples for further mechanical research.

In order to observe the behaviour of EPS20 samples subjected to underpressure and then placed under water, one experiment was performed in a small air-tight glass pot. In this way it was possible to observe the deformation of the samples in each phase of the experiment. The observation showed that deformation of the samples did not occur in any of the following cases: when subjected to an underpressure, when water was passed into the kettle, when the sample was kept completely under the water, when the underpressure after immersion was kept on, after the air was allowed to enter the glass pot, and after the inside pressure became equal to atmospheric pressure. However, when the samples were taken out of the water, the deformation, a reduction of the original sample volume and contraction of the surface cells occurred at once.

This behaviour could be explained by taking into account a pressure difference created between the pressure deeper inside the sample and pressure within the outer cells, leading to tensile forces in the outer layer. The visible effect is a deformation of the whole sample, a reduction of the total volume and a contraction of the cells at the sample’s surface.

All in all, the investigation pointed out the fact that if the cell structure has been damaged the water absorption of EPS would increase significantly.

5.3.4 Stimulating water absorption by freeze-thaw cycles

Absorption of water by EPS can be accelerated by cyclic exposure of this material to water vapour and low temperature respectively. The explanation could be provided by analysis of the transport mechanism responsible for water absorption of EPS. This seems to be diffusion combined with capillary condensation. At low temperatures capillary condensation of the water vapour penetrated by diffusion into the EPS cells is accelerated. Consequently, the vapour concentration inside the cells is reduced. The result is an increase of the difference in relative humidity inside and outside the cells after removing the samples from the freezer and placing them back in the space saturated with vapour. The higher concentration difference stimulates the diffusion process and vapour penetrates rapidly through the thin cell walls. In this way, the quantity of water absorbed by EPS increases by a certain amount during each cycle, and repeated freeze-thaw cycles result in water absorption by the EPS in a shorter time than in the case of immersing EPS.

The test procedure consisted of two steps. Firstly, EPS samples were held in a space saturated with water vapour and then, after a few hours, they were placed in a freezer.
(EPS20) or in a freezing chamber (EPS15) and held there for a few hours. The saturated space, where the EPS20 samples were placed in the first phase of each cycle, was created in a closed large kettle. The kettle contained a layer of approx. 0.2 m water which was continuously heated, keeping the temperature at a constant value of 50°C. The temperature maintained in the freezer and the freezing chamber was -25 and -12°C respectively. In total about 30 cycles were carried out, over a period of 15 days.

The intensity of water absorption, stimulated by freeze-thaw cycles, depends on the sample dimensions. The relatively greater surface of small samples gives vapour direct access to a relatively larger number of surface pores per volume unit. This manifests itself in a faster water absorption by the smaller samples. In any case, the dimensions of both EPS20 and EPS15 samples were not optional. Samples had to be appropriate for the uniaxial and compression test apparatus. Consequently, the EPS20 samples had a cylindrical form with a diameter of 100 mm and a height of about 200 mm, while the EPS15 cylinders were 300 mm high with a diameter of 150 mm, identical to those used in the previous experiments.

The average absorption value for EPS20 was equal to 1.1% v/v after approx. 30 cycles. It is a lower level than the water absorption level obtained by means of immersion (average 1.54% v/v; it should be taken in account that the immersion took over a year). The results for EPS15 were lost. By error, the cooling in the freezing chamber with the EPS15 samples was turned off and the samples dried after the test. Nevertheless, the most important fact is that the samples were exposed to freeze-thaw cycles. Through this treatment representative EPS20 specimens were provided for the determination of the combined influence of extreme temperatures and absorbed water on their mechanical behaviour. The EPS15 cylinders were representative only for the dry material state after exposure to freeze-thaw cycles. The EPS in the sub-base, surrounded by wet subsoil and containing water, may be periodically subjected to freeze and thaw, the combination of conditions simulated by the treatment performed in these tests.

The temperatures calculated in the sub-base of EPS for average Dutch climatic conditions varied between -4°C and 27°C during a year with an average summer and winter. Undoubtedly, more extreme temperature values will occur. It is, however, not likely that the temperatures in an EPS sub-base are lower than -12°C or higher than 50°C. Accordingly, the test temperature intervals were extreme and covered practical conditions well. An unchanged material behaviour will be a solid guarantee that no problems should be expected with respect to the mechanical behaviour of EPS built-in in a sub-base of a road pavement structure.

### 5.3.5 Concluding remarks on water absorption

EPS absorbs water very slowly and in a limited quantity. The average absorption of water by immersion was equal to 1.54% v/v after one year at average ambient temperature of approx. 18°C. The asymptotic value for EPS20 seems to be equal to 2% v/v. For EPS15 the absorption was 1.56% after more than two months under water. In the
beginning, the absorption is relatively fast but after a month the increase becomes almost unnoticeable. However, these values are much lower than the values described in the literature (varying between 3% v/v for EPS30 and 5% for EPS15), although these were obtained during investigations on EPS as an insulating material implying that the measurements were carried out on samples in the shape of boards. A board has a much bigger surface-to-volume ratio than the cylinders used in the experiments described here. This underlines that the reported level of water absorption depends upon the shape and the surface conditions of the samples.

Subjecting EPS20 samples to an underpressure resulted in an accelerated absorption of water. A higher value of the underpressure results in a faster water absorption. Unfortunately, subjecting specimens to underpressure damages the samples, making this procedure inconvenient for preparing samples for further mechanical research. However, the investigation pointed out that the EPS will absorb significantly more water if the cell structure is damaged for some reason.

The absorption of water by exposing EPS20 samples to freeze-thaw cycles was equal to 1.1% v/v after approximately 30 cycles. Although this indicates that the water absorption by EPS could be accelerated to a certain extent under such severe conditions and a large increase in weight might occur in a relatively short period of time, it is not likely to be of major importance for the application of EPS in pavement engineering. This is because the severe conditions as used in this particular test will not occur in road sub-bases.

5.4 COMPRESSION TESTS

Compression tests on EPS15 and EPS20 were performed to provide a global insight into the stress-strain behaviour of EPS in compression, when loaded up to the stress and strain levels which widely exceed the elastic region. Knowing the stress-strain relation within such a wide range, the boundaries of the linear-elastic region can be determined as well as the development of the modulus of the tested EPS types. To investigate whether loading speed, specific state of material or low temperatures affect the behaviour of EPS15 and EPS20 these materials were subjected to compression at different strain rates in dry, wet and freeze-thaw-treated state, at normal and low temperatures. The compression tests done at room temperature will be discussed firstly followed by the tests performed at low temperatures.

5.4.1 Compression tests at normal temperature

5.4.1.1 Test specifications

The maximum strain to which the EPS specimens were compressed in the first of the two test series was about 10%. To determine the effect, if any, of the strain rate on the stress-strain behaviour of EPS, the compression was carried out strain-controlled at four
loading speeds. The loading speeds were equal to 0.2, 1, 10 and 100 mm/s which corresponded to strain rates of 4, 20, 200 and 2000 %/min. The minimum and maximum applied compressing speed were dictated by the limitations of the used device; the remaining loading speeds are intermediate values.

In the second test series EPS specimens were subjected to force-controlled compression up to five selected stress levels chosen to be close to the boundaries of linear-elastic region (determined in the previous compression tests) for the sake of establishing limit stress and strain values under which EPS behaves elastically.

A picture of the test set up with an EPS specimen is given in Figure 5.5. Figure 5.6 shows the upper loading plate with two symmetrically placed displacement transducers. As measurement result the average vertical deformation obtained during compression was taken. Each test was performed with three EPS specimens, 300 mm high cylinders with a diameter of 150 mm, taken from different block positions. The intention was to eliminate the influence of the systematic variation in density (see section 5.7) on the test results.

Figure 5.5 - Compression test set-up
Figure 5.6 - Upper part of the compression test set-up with two displacement transducers

EPS15 was tested in wet and dry conditions. The preparation procedures of the wet EPS15 specimens by both immersion and exposure to freeze-thaw cycles have been described in paragraphs 5.3.2 and 5.3.4 respectively. Average water absorption among the immersed EPS15 specimens amounted to 1.56% v/v. By subjecting both wet and dry EPS15 samples to compression the influence of water absorption, if any, on the behaviour of the material was determined.

EPS20 was compressed only in dry state since the experiments done earlier [5] showed that a somewhat higher modulus could be expected if the EPS20 samples do
contain water. Accordingly, EPS20 was subjected to compression in the more critical dry state.

5.4.1.2 Influence of strain rate on EPS behaviour

The resulting stress and strain values obtained during the compression tests on dry EPS15 for four loading speeds, namely 0.2, 1, 10 and 100 mm/s (corresponding to strain rates of 4, 20, 200 and 2000%/min) are graphically presented in Figure 5.7. Each curve represents average results obtained from three samples, eliminating the influence of variations in EPS density on the measurement results. In Figure 5.7 a blow-up of the stress-strain curves in the strain range up to 1% is shown.

![Stress-strain diagram of dry EPS15 under compression including a blow-up within a strain range of 1%, with four different loading speeds; each curve represents the average test result of three samples.](image)

The comparison of the curves shows that, as in many other materials, the loading speed has an influence on material behaviour of dry EPS15 under compression. The measured stress values of dry EPS15 under compression decrease with decreasing loading speed. It would be more accurate to say that the above-mentioned influence becomes really obvious beyond the strain range of 1%. Within the strain range of 1% the loading speed does not seem to affect the EPS15 behaviour; the presented curve found for the loading speed of 100 mm/s deviated somewhat from the general trend.

The compression test results on wet EPS15 are graphically presented in Figure 5.8. With regard to the relationship between the strain rate and the stress-strain behaviour of wet EPS15 identical conclusions could be drawn as for dry EPS15.
Figure 5.8 - Stress-strain diagram of wet EPS15 under compression with four different loading speeds; each curve represents the average data of three samples.

The stress-strain diagrams for dry EPS20 specimens compressed with loading speeds of 0.2, 1, 10 and 100 mm/s are shown in Figure 5.9.

Figure 5.9 - Stress-strain diagram of dry EPS20 under compression with four different loading speeds; each curve represents the average data of three samples.

Both EPS15 and EPS20 show plastic yielding if the deformation goes beyond 2%. This is apparently associated with failure of the EPS cell structure. The initial failure of an EPS specimen occurs probably locally. Continuing compression leads to the collapse of the neighboring EPS cells and the initial failure zone spreads. The closed air inside the cells resists an additional deformation of cells till the moment of cell failure. After the cells collapse, the released air escapes. In the case of higher displacement-controlled
loading speeds the air has less time to escape from the EPS. This could result in some what stiffer EPS behaviour compared to compression at lower loading speeds.

The only existing quality control specification concerning the mechanical properties of EPS20 used in the sub-base is related to its minimal compressive strength. The minimal required compressive strength is 100 kPa at 5% deformation [7]. The test has to be performed on samples (50×50×50 mm) which, because the bulk density can vary within an EPS block, have to be taken from three different locations from the block. The samples should be in dry condition before compression. As already mentioned, the prescribed rate of compression is 1 mm/min (2%/min). Compression test results described in this paragraph point out that the prescribed deformation of 5% reflects the EPS behaviour after occurring of plastic yielding, i.e. local failure of the cell structure. Furthermore, the prescribed loading speed is very low and guarantees that in such a way measured EPS compressive strength can only be higher under normal traffic loading conditions.

Summarizing the test results it can be stated that loading speed affects the EPS material behaviour under compression. EPS yield strength increases with increasing loading speed. The influence of loading speed, however, seems to be very limited, if not negligible, in the strain range up to 1%. Beyond strains of 1 to 2% EPS offers a large plastic strain capacity over which the load bearing capacity is maintained.

5.4.1.3 Influence of water absorption on EPS15 behaviour

Influence of water absorption on EPS15 mechanical behaviour under compression was tested by exposing EPS in dry, wet and freeze-thaw-treated state and comparison of the corresponding results. These results are summarized in Figure 5.10. All EPS15 specimens which have been in contact with water during the preparation period for the tests were assigned as wet.

![Figure 5.10](image)

Stress-strain diagram of dry and wet EPS15 under compression with four different loading speeds, namely 0.2, 1, 10 and 100 mm/s
From this figure it appears that water absorption has no negative effect on material behaviour of EPS15 under compression. The wet specimens demonstrated even a better resistance against deformations than the dry specimens. Such a conclusion was valid for all performed loading speeds. The water vapour which diffused inside the cells seems to contribute to an increase of the rigidity of the cells resulting in a somewhat stiffer material behaviour.

Immersion of the wet EPS15 cylinders previously compressed to a strain level of 10%, resulted in absorption of a relatively large amount of water. Average water absorption measured few minutes after immersion was higher than 4% by volume. It seems that after the cell structure is damaged, liquid water penetrates faster into the EPS and accumulates not only in small voids between the fused cells but in the collapsed cells as well. This causes a multiplication of the maximum water content in the EPS when deformed beyond the failure limit of its cell structure. According to the conclusion drawn in paragraph 5.4.1.2 the failure of the EPS cell structure seems to occur in both EPS15 and EPS20 if deformation becomes greater than 2%.

5.4.1.4 Evolution of tangent modulus

If in the previously presented figures the strain scale is divided in uniform strain intervals, $\Delta e$, and for each strain interval a corresponding stress interval $\Delta \sigma$ is determined, a local tangent value of the material modulus $E_t = \Delta \sigma / \Delta e$ can be computed. In this way the evolution of the local tangent modulus $E_t$ can be drawn corresponding with the stress-strain diagrams obtained under compression. This material modulus has a constant value within the linear-elastic region corresponding with the linear-elastic modulus $E$. During further compression the modulus value $E_t$ decreases as a result of material softening. Cell structure of EPS starts to collapse gradually, manifested by a fast increase of the strain value as the stress slightly increases. In the stress-strain diagrams this is evident from the decreasing gradient of the $\sigma$-$e$ curves.

The computed discrete values of the local tangent modulus $E_t$ versus total strain values $e$ are drawn in Figures 5.11 and 5.12 for EPS15 in both dry and wet state. The results represent average outcomes of three EPS15 specimens minimizing the effects of density variations. The presented $E_t$-$e$ curves only cover the strain range up to 2.4%. Within this region the value of $E_t$ after being constant in accordance with linear-elasticity at the beginning of loading, decreases quite markedly starting from 0.4 à 0.5%.

Since the $E_t$ is constant in the strain range up to 0.4 à 0.5%, the 0.4 à 0.5% limit is called the linear-elastic limit for EPS15.
The $E_\tau\cdot\varepsilon$ curves obtained in uniaxial compression test on dry EPS20 specimens are shown in Figure 5.13. Higher $E_\tau$-values are found within the linear-elastic region ($E_{EPS}$-values) than in case of EPS15 but in general an identical conclusion can be drawn. Briefly, the modulus decreases very fast once the obtained deformations exceed the linear-elastic boundary which is again at 0.5%.
Figures 5.14 to 5.15 clearly show a significant reduction in resistance to deformations of EPS15 and EPS20 when the material is loaded above its linear-elastic limit. Such a change in EPS behaviour does not proceed gradually but dramatically once its cell structure is damaged. In practice it implies the need for caution in handling of EPS blocks in the construction phase. Overloading the EPS sub-base in the construction phase due to construction traffic would lead to a lower $E_{\text{EPS}}$ value in service. Therefore, the maximum linear-elastic strain must not be exceeded in the EPS blocks implemented in a sub-base.

### 5.4.1.5 Effect of variation in density on EPS linear-elastic region

In the previous paragraphs the measurement results based on three EPS specimens were consequently averaged before presenting them. This is done to eliminate the effects that the variety in EPS bulk density could have on the interpretation of the results. To what extent the variation in density of the EPS samples influences the material behaviour is illustrated by means of two examples given in Figures 5.16 and 5.17.

![Fig. 5.14](Image)

**Fig. 5.14** - Stress-strain diagrams of three dry EPS15 samples under compression with loading speed of 1 mm/s; three curves represent measured values while the fourth curve represents the average data

![Fig. 5.15](Image)

**Fig. 5.15** - Stress-strain diagrams of three dry EPS20 samples under compression with loading speed of 10 mm/s; three curves represent measured values while the fourth curve represents the average data

There are three $\sigma$-$\epsilon$ curves obtained for dry EPS15 and EPS20 samples respectively, presented together with the corresponding averaged $\sigma$-$\epsilon$ curves. The loading speed, EPS type as well as the hygric state of material were identical for all of them. Still, a significant difference in behaviour under compression was registered. All this despite the fact that most of them were cut from the same EPS block.

It may be expected on the basis of the figures that the variation in EPS density within a block affects both the boundaries of linear-elastic region of EPS and its behaviour in this region. In Chapter 3 the stress levels expected in EPS used in the sub-base under
traffic loading were highlighted. If the maximum stresses in EPS have not exceeded the linear-elastic boundary, permanent deformations cannot occur. Therefore it is necessary to investigate to which extent the variety in EPS density leads to a change of the linear-elastic stress ($\sigma_{le}$) and strain ($\varepsilon_{le}$) limit values. This investigation was carried out, firstly, by determination of the linear-elastic stress and strain boundaries for each compressed EPS specimen separately and, secondly, by analyzing the relations between $\sigma_{le}$ and density, $\varepsilon_{le}$ and density and the linear-elastic modulus of elasticity ($E_{EPS}$) and density.

Figure 5.16 shows boundary stress values of the linear-elastic region versus the dry density of the EPS specimens under compression (either in dry or wet conditions). A certain relationship between $\sigma_{le}$ values and EPS density appears.

The corresponding boundary strain values versus specimen density are graphically presented in Figure 5.17. From this figure it is evident that the EPS ultimate linear strain is indifferent to both the state of material (dry, wet or freeze-thaw-treated) and the strain rate (0.2, 1 and 10%/s) up to the strain value of approximately 0.4%. Following the figures the conclusion can be drawn that neither water absorption nor treatment by freeze-thaw cycles have an influence on the EPS material behaviour. By contrast, the boundary stress values show a marked roughly linear-dependency on the (dry) density (Figure 5.16).

![Figure 5.16](image1.png)  
**Figure 5.16**  
Boundary stress values of linear elastic region vs. corresponding densities of the EPS specimens

![Figure 5.17](image2.png)  
**Figure 5.17**  
Boundary strain values of linear elastic region vs. corresponding densities of the EPS specimens

As the next step the modulus of elasticity of the EPS, $E_{EPS}$, was calculated from the previously discussed $\sigma_{le}$ and $\varepsilon_{le}$ values and presented with the belonging $\rho$ values as shown in Figure 5.18. From this graph a clear relationship appears between the elasticity modulus and the (dry) density of EPS.
By performing regression analysis on the test results the relationship $E_{\text{EPS}}\rho_{\text{EPS}}$ was quantified. The curve found to fit data was the following power function:

$$E_{\text{EPS}} = 0.1284\rho^{1.368}$$  \hspace{1cm} [eq. 5.1]

Summarizing the test results, the boundary of the linear-elastic region, found for dry and wet EPS15 samples under compression ranged between 0.37 and 0.60%. The corresponding limit stresses were within the range 21.4 to 34.7 kPa. The associated E modulus varied between 4.9 and 6.0 MPa. There was no significant difference found between the behaviour of dry and wet EPS15 samples. The boundary strain values for EPS20 related to linear-elastic region ranged between 0.43 and 0.53%. The stress values varied between 36.0 and 60.0 kPa while the resulting E moduli have values between 7.37 and 9.27 MPa.

### 5.4.2 Compression tests at low temperature

The behaviour of a material can be affected by low temperatures. A material can become more stiff and brittle at lower temperatures. Polystyrene, the basic material from which EPS is produced, is such a material. However, the material is used at temperatures below the glass-transition temperature (102°C) so the changes in material behaviour by going from room to freezing temperature are not expected to be very dramatic. Still, the cell walls are made of polystyrene which means that the mechanical behaviour could be affected to a certain extent by low temperatures occurring in the sub-base. This possibility points to the necessity for testing EPS at temperatures at least as low as the lowest that can occur in the sub-base. In the scope of investigation both monotonic compression tests and cyclic loading tests on EPS20 were carried out at temperatures down to -15°C. In this sub-section the monotonic compression tests are discussed.
5.4.2.1 Test specifications

In this test series the stress-strain behaviour of EPS20 under compression was investigated. The small-strain behaviour, particularly the linear response, is of importance for the application of EPS in the sub-base. Therefore, the tested EPS20 specimens were loaded up to the strain level of 3%. The specimens were EPS20 cylinders 200 mm high with a diameter of 100 mm.

The mode of loading was strain-controlled and the loading speed was equal to approximately 14 mm/min corresponding to a strain rate of 7 %/min; this level was dictated by the device limits. A relatively simple loading frame with an electric motor was utilized because the use of advanced servo-hydraulic equipment at a test temperature lower than -10°C was judged to be too risky. The stresses during the compression tests were observed indirectly through observation of the force measured by a force meter mounted under the plate with the sample. Exact values of the deformation were determined from the set deformation speed and the duration of the tests.

5.4.2.2 Linear-elastic region of EPS20 at low temperature

A design criterion for pavement structures with an EPS20 sub-base prescribes the maximum allowable strain in the EPS layer under representative traffic loadings. In subsection 5.5.1 we have determined the boundary of the linear-elastic region for EPS15 and EPS20. In this sense it is of importance to determine the influence, if any, of low temperatures on the linear-elastic behaviour of EPS20. For this purpose, the linear-elastic region of EPS20 was determined in this test series by subjecting dried specimens to compression at temperatures ranging between -8.6 and -12.9°C. An example of a resulting stress-strain curve is given in Figure 5.19.

![Figure 5.19](image_url)

Stress-strain curve of EPS20 subjected to compression at T = -11.2°C
A 'soft' behaviour of EPS was observed at the beginning of all compression tests. It was caused by lack of parallelism between the surface of the loading plate and the sample surface, i.e. when the contact was only between the loading plate and part of the sample surface; Further loading resulted in full contact and by consequence in a stiffer behaviour of EPS. In order to define the linear-elastic region, a straight line was fit through the initial portion of the registered force-time curve as shown in Figure 5.19.

The linear-elastic region, found for EPS20 samples under slow compressive loading (strain rate 7%/min) at low temperatures, ranged between 0.51 and 0.84%. The corresponding stresses are within the range of 45.9 to 51.0 kPa. The associated E values varied between 6.0 and 10.0 MPa. The lower values of the E modulus were calculated for the larger linear-elastic regions while the larger E values were measured for the shorter strain intervals: E-value of 6.0 MPa corresponds to the strain interval \( \Delta \varepsilon_k = 0.84\% \) and E-value of 10.0 MPa to \( \Delta \varepsilon_k = 0.51\% \). The complete results of the compression tests at low temperatures are listed in Table 5.1.

| Linear-elastic region of EPS20 at low temperatures |
|-----------------|--------|----------|----------|--------|
| \( \rho_{dry} \) [kg/m³] | Temp [°C] | \( \Delta \sigma_{le} \) [kPa] | \( \Delta \varepsilon_{le} \) [%] | \( E_{le} \) [MPa] |
| 18.87 | -12.9 | 51.0 | 0.84 | 6.0 |
| 19.02 | -11.2 | 46.9 | 0.63 | 7.5 |
| 19.52 | -10.0 | 47.2 | 0.60 | 7.9 |
| 20.42 | -10.6 | 45.9 | 0.66 | 7.0 |

Table 5.1

Linear-elastic region for EPS20 under compression at low temperatures

From a comparison of the test results given in Table 5.1 with those given in paragraph 5.4.1.5 it appears that low temperatures have no negative impact on the mechanical behaviour of EPS20. The boundary stress and strain values of the linear-elastic region under slow compressive loading and the associated moduli of elasticity determined under these conditions were slightly higher than the corresponding values found at room temperature.

5.4.3 Concluding remarks on compression tests

Strain rate affects the EPS material behaviour under compression. The yield stress of EPS increases with increasing strain rate, i.e. loading speed. The influence of strain rate on the modulus, however, seems to be limited in the strain ranges from 0 to 1%.

The linear-elastic region, found for dry and wet EPS15 samples under compression, ranged between 0.37 and 0.60%. The corresponding stresses were within the range 21.4 to 34.7 kPa. The associated E modulus varied between 4.9 and 6.0 MPa. The boundary strain values for EPS20 related to the linear-elastic region ranged between 0.43 and
0.53%. The stress values varied between 36.0 and 60.0 kPa while the resulting E modulus has a value between 7.37 and 9.27 MPa.

There is a significant reduction in resistance to deformation of both EPS15 and EPS20 when the material is loaded above its linear-elastic limit. Such a change in EPS behaviour does not occur gradually but dramatically once its cell structure is damaged. In practice it implies the need for caution in handling the EPS blocks during the construction phase. Overloading the EPS sub-base in the construction phase by construction traffic would lead to a lower $E_{\text{EPS}}$ value in service. Accordingly, the actual $E_{\text{EPS}}$ modulus would be lower than the input $E_{\text{EPS}}$ value used for pavement design. Therefore, the maximum linear-elastic strain must not be exceeded in the EPS blocks used in a sub-base. In that case the elasticity modulus of EPS has a constant value.

Both EPS15 and EPS20 show plastic yielding if the deformation becomes greater than 2%, at that stage failure of the EPS cell structure occurs. The initial failure of an EPS specimen occurs probably locally. Continuing compression leads to the collapse of the neighboring EPS cells and the initial failure zone increases. The air pressure established by the air closed inside the cells resists an additional deformation of cells till the moment of cell failure. After the cells collapse the released air escapes. In the case of higher deformation rate the air has less time to escape from the EPS, and will contribute to the resistance against compression. This could result in a somewhat stiffer EPS behaviour at higher strain rates when compared to the values obtained at lower strain rates.

Failure of the EPS cell structure caused by overloading, leads to a higher water absorption capacity compared to undamaged EPS blocks. Therefore the actual density of the saturated EPS blocks previously overloaded, e.g. in the construction phase, could exceed the reference weight of 100 kg/m³ used for saturated EPS in the weight-balance calculations.

Neither water absorption nor treatment by freeze-thaw cycles showed to have negative effects on the material behaviour of EPS15 under compression. The wet specimens showed even better resistance against deformations than EPS15 in dry state.

A clear relationship appears between the elasticity modulus and the density of EPS. The observed relationship $E_{\text{EPS}} \rho_{\text{EPS}}$ can be approximated by the following equation:

$$E_{\text{EPS}} = 0.1284 \rho_{\text{EPS}}^{1.368}$$  \hspace{1cm} [eq. 5.1]

Low temperatures have no negative impact on the mechanical behaviour of EPS20. The boundary stress and strain values of the linear-elastic region under slow compressive loading and the associated moduli of elasticity determined under these conditions were not lower than the corresponding values found at normal temperature.
5.5 CREEP TESTS

The dead weight of the pavement layers above the EPS sub-base provides a constant load on the EPS. The stress occurring in the EPS, resulting from the loading by dead weight, is classified as the static stress component, $\sigma_s$, in section 5.6. The corresponding negative strain of the static stress component in the material increases in time, under unchanged value of $\sigma_s$, what is defined as creep. Theoretically, the material continues to deform at constant stress for an infinite period of time. However, if the temperature is much lower than the absolute material glass transition temperature, the effect of creep is negligible after some time.

A practical consequence of this phenomenon for EPS in a pavement structure is the increase of the vertical permanent deformation of this layer in a certain period of time after completion of the construction of the pavement. Therefore, it is necessary to investigate creep to enable the prediction of the corresponding ultimate settlement of the pavement structure because of creep in the EPS sub-base.

5.5.1 Test characteristics

For determination of the EPS creep range under the representative loading two test series were carried out. In the first series only EPS20 cylinders exposed to a single stress level were tested. In the second series creep of both EPS15 and EPS20 samples was measured under two different stress levels. For both test series only the EPS samples in dry conditions were used.

The experimental set up in the first test series for the determination of creep in the EPS20 is shown in Figure 5.20. Dead weight of the layers above the EPS sub-base was simulated by means of a load placed on top of the cylindrical EPS samples ($\Omega 200$ mm, $H = 100$ mm). The load weight was chosen in such a way that the vertical stress was about 20 kPa. The increase of the vertical deformation of the sample was measured by means of a micrometer on top of the load.

In the second test series creep was measured for EPS15 as well as for EPS20 ($\Omega 150$ mm, $H = 300$ mm) under the two different loads. The first load (18 kg) resulted in a stress of 10 kPa in EPS, corresponding to the effects of relatively thin upper layers laying at top of the EPS sub-base. The second applied load caused a stress of 20 kPa in the EPS specimens representing a relatively thick pavement structure (e.g. 0.25 m asphalt concrete + 0.6 m unbound base + 0.1 m capping concrete layer).

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Figure 5.20
Set up for the determination of creep of small EPS20 cylinders
The initial deformation value and the vertical deformations of the EPS15 and EPS20 specimens after loading were measured by means of a cathetometer connected to a micrometer scale as illustrated in Figure 5.21; for this purpose a reference point was drawn near to the top on each of the EPS cylinders. The actual values of the obtained vertical deformation at a certain moment were further determined by measuring the vertical displacements of those reference points. The set up was placed in a conditioned room with a controlled temperature (±21°C) and humidity (±50%).

The use of the cathetometer provided two advantages: firstly, it enabled the testing of a dozen samples simultaneously (just by rotating the cathetometer); secondly, the measurement of the initial vertical deformation after loading became possible. However, this measuring method was less accurate than direct reading of the micrometers. Furthermore, by observing just a single reference point per specimen the inclination of the upper cylinder side under the condition of a not perfectly centralized load was ignored.

5.5.2 Creep under representative loading

Figure 5.22 illustrates creep of EPS20 during a period of more than a year. After more than 400 days, the vertical deformation kept on increasing. This increase is much better visible in Figure 5.22 with a logarithmic time scale. The end of the (very slow) creep process cannot be recognized. Nevertheless, it can be expected the ultimate creep values will stay beyond 0.4%.
Creep of EPS20 subjected to a static load which corresponds to the heavy top layers ($\sigma_s = 20$ kPa) would only be a few tenths of a percent. It is in agreement with literature [8, 9] where very small creep values are reported for similar stress levels. A considerable part of the creep, about 0.07% of 0.20%, occurs already within the first day. A practical consequence is a very small additional settlement of the pavement structure in time due to creep of the EPS sub-base layer.

In the literature [10] the relationship $\epsilon_s = a t^n$ including the empirical parameters $a$ and $n$ is proposed for creep strain at block-molded EPS. This equation, with time $t$ expressed in hours is based on the approach used by Fyndley as described in [11]. The values for the parameters $a$ and $n$ which are given in literature [10] do not fit, however, the results shown in Figure 5.22.

### 5.5.3 Immediate strain and creep under representative load

The total strain in block-molded EPS due to the dead weight of the top layers in a pavement structure, consists of two components:

$$\epsilon_t = \epsilon_i + \epsilon_c \quad \text{[eq. 5.2]}$$

where:
- $\epsilon_t$ - total strain [%]
- $\epsilon_i$ - immediate strain [%]
- $\epsilon_c$ - creep strain [%]

The total vertical strains of both EPS15 and EPS20 due to the static stress of 10 and 20 kPa, respectively, are shown in Figure 5.23. In order to get the results representative
for the EPS blocks the strain values were averaged per three specimens, identically with previous compression and cyclic tests. Per set the specimens with a high, average and low density regarding the block density were tested.

![Graph showing immediate strain and creep due to a static stress](image)

**Figure 5.23** Immediate strain and creep curve of EPS15 and EPS20 due to the loading which simulates the dead weights of both the 'light' and 'heavy' toplayers in the pavement structure with a log time scale

Total vertical strain to be expected in the EPS sub-base due to the dead weight has a level up to approximately 0.5% in the case of (very) heavy pavement structures. The corresponding strain range amounted to approximately 0.25% for the two times lighter toplayers. The immediate vertical strain (before creep obtained) had values of approximately 0.3% and 0.15% for the static stress of 20 and 10 kPa respectively. Creep seemed to be linear, i.e. approximately two times lower when a twice lighter dead weight was applied. A significant part, about 50%, of creep was obtained already within one day after loading. As far as the additional vertical deformation in time due to the further creep of the EPS layer is concerned it could be estimated at 0.1% to 0.2% by extrapolation up to 20 years, thus of minor importance for practice.

A significant difference between creep of EPS15 and EPS20 was not observed. Slightly higher creep was measured for EPS15 under both static applied stress values (10 kPa and 20 kPa). The measured small variations in vertical strain during the creep test could be explained by previously mentioned potential inaccuracies of the measurements.
5.5.4 Concluding remarks on immediate strain and creep

Total vertical strain in the EPS sub-base due to the dead weight of the top layers has a range of approximately 0.5% in the case of very heavy pavement structures. The immediate vertical strain in EPS blocks, a significant part of the total vertical strain, amounts to approximately 0.3%.

The creep level of EPS20 caused by a static stress of 20 kPa is rather limited and seems to be less than 0.2% after more than a year. Creep seems to be linear for both EPS15 and EPS20. About half of the expected maximum creep occurs already within the first day. All in all, the additional settlement of the pavement structure due to creep in the EPS sub-base will be rather limited, in order of a few tenth of a percent. Therefore, this deformation increase is of minor practical importance for pavement design.

5.6 UNIAXIAL CYCLIC LOADING TESTS

5.6.1 Introduction

The primary shortcoming of the monotonic compression tests, discussed in the previous subsection is that these conditions differ substantially from those to be expected in the EPS sub-base. The sub-base loading conditions are characterized by both dead weight of the overlaying layers and the dynamic loads due to the traffic passing above. When testing materials the adequate representative test loading should therefore consist of two stress components, a static stress simulating the dead weight, and a cyclic stress component simulating the traffic loading. In order to subject EPS20 and EPS15 samples to more representative loadings, uniaxial cyclic loading tests were carried out on these EPS types.

The used cyclic loading apparatus made it possible to apply loadings with a frequency of 3 and 6 Hz in order to approximate the actual loading due to the traffic at the depth of the EPS blocks (see Chapter 3). Here again, as in the compression tests, EPS15 and EPS20 specimens were in three different states, namely dry, wet and treated by freeze-thaw cycles. The uniaxial cyclic loading tests were also performed at a low temperature.

The performed uniaxial cyclic loading tests can be classified into three series. First, tests were performed on small cylindrical EPS20 samples (Ø 100 mm, H = 200 mm) by means of a small loading device. Secondoy, uniaxial cyclic loading tests on both EPS15 and EPS20 using bigger cylindrical samples (Ø 150 mm, H = 300 mm) with a more sophisticated measuring device. Finally, small EPS20 cylinders (Ø 100 mm, H = 200 mm) were subjected to strain-controlled cyclic loading test at low temperatures. Test specifications and results for each of the three mentioned test series will be discussed in the separate subsections.
5.6.2 Cyclic loading tests on small EPS20 specimens

5.6.2.1 Test specifications

In order to subject EPS20 samples to representative loads the uniaxial tests were carried out by means of an advanced cyclic loading apparatus, schematically shown in Figure 5.24. By using this apparatus a short sinusoidal pulse could be applied which simulated the load pulse shape caused by a moving wheel. The possibility for applying a short pulse (up to 10 Hz) is important since the representative loading frequency has a value between 3 and 6 Hz. Furthermore, the static and cyclic dynamic stresses could be selected as desired, creating conditions very similar to those in pavement structures. Inside the loading cell of the used apparatus there was a possibility to apply an all-around confining stress. However, there was no need to apply this confining stress because the calculated values for the horizontal stress component in the EPS sub-base layer were negligible. The test was therefore reduced to an unconfined compression test. This is on the conservative side because a confining stress could only contribute to a better EPS stress-strain behaviour. All in all, the cyclic load apparatus generates loadings on EPS samples, close to the representative values with respect to pulse frequency, pulse shape, and static and dynamic stress components.

Two test subseries were distinguished, The fist subseries contained the tests performed to determine the dynamic elasticity modulus, $E_{\text{dyn}}$, and Poisson’s ratio, $\nu$, for EPS20 under cyclic stresses assumed to be representative for sub-base conditions. The second series concerns cyclic loading tests where the specimens were loaded by a step-wise increase of the stresses up to a total stress of 100 kPa. In this way the EPS stress-strain behaviour could be studied in a more general way.

The analysis of the test results in this series was limited to the dynamic stress and strain values due to the cyclic loading. Dynamic modulus of elasticity and the Poisson’s ratio were determined. The radial strains, needed to calculate $\nu$, were measured by a set of three proximity transducers, mounted in a horizontal plane at half the sample height.

5.6.2.2 $E_{\text{dyn}}$ modulus and Poisson’s ratio of EPS20

The stress conditions that were used in the repeated load uniaxial test to determine $E_{\text{dyn}}$ and the Poisson’s ratio were as follows. The applied static stress component ($\sigma_s$) ranged from 15 to 30 kPa while the cyclic stress ($\sigma_c$) varied between 30 and 35 kPa. The loading frequency was 3 or 6 Hz. The number of load repetitions varied between 32,400 and 270,000 per test. All the test results are listed in Table 5.2.
### Table 5.2 - Resulting EPS material parameters from uniaxial cyclic tests for various conditions

<table>
<thead>
<tr>
<th>Sample State</th>
<th>$\rho_{\text{dry}}$ [kg/m$^3$]</th>
<th>$\sigma_{\text{static}}$ [kPa]</th>
<th>$\sigma_{\text{cyclic}}$ [kPa]</th>
<th>freq. [Hz]</th>
<th>Loading cycles</th>
<th>Vert. $\varepsilon_{\text{dyn}}$ [%]</th>
<th>Radial $\varepsilon_{\text{dyn}}$ [%]</th>
<th>$E_{\text{dyn}}$ [MPa]</th>
<th>$\nu$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>18.83</td>
<td>25</td>
<td>30</td>
<td>6</td>
<td>84,000</td>
<td>0.36</td>
<td>0.039</td>
<td>8.3</td>
<td>0.11</td>
</tr>
<tr>
<td>Dry</td>
<td>18.83</td>
<td>25</td>
<td>30</td>
<td>3</td>
<td>+30,000</td>
<td>0.39</td>
<td>0.040</td>
<td>8.3</td>
<td>0.10</td>
</tr>
<tr>
<td>Dry</td>
<td>19.80</td>
<td>15</td>
<td>30</td>
<td>3</td>
<td>32,400</td>
<td>0.39</td>
<td>0.035</td>
<td>8.0</td>
<td>0.09</td>
</tr>
<tr>
<td>Dry</td>
<td>19.33</td>
<td>30</td>
<td>33</td>
<td>6</td>
<td>84,000</td>
<td>0.48</td>
<td>0.044</td>
<td>6.9</td>
<td>0.09</td>
</tr>
<tr>
<td>Dry</td>
<td>19.47</td>
<td>30</td>
<td>35</td>
<td>3</td>
<td>270,000</td>
<td>0.47</td>
<td>0.042</td>
<td>7.4</td>
<td>0.09</td>
</tr>
<tr>
<td>Wet</td>
<td>19.02</td>
<td>25</td>
<td>30</td>
<td>3</td>
<td>48,000</td>
<td>0.47</td>
<td>0.034</td>
<td>6.4</td>
<td>0.07</td>
</tr>
<tr>
<td>Wet</td>
<td>19.15</td>
<td>30</td>
<td>35</td>
<td>3</td>
<td>212,000</td>
<td>0.55</td>
<td>0.042</td>
<td>6.4</td>
<td>0.08</td>
</tr>
<tr>
<td>Wet</td>
<td>19.32</td>
<td>25</td>
<td>30</td>
<td>3</td>
<td>224,000</td>
<td>0.57</td>
<td>0.032</td>
<td>8.1</td>
<td>0.09</td>
</tr>
<tr>
<td>Dried</td>
<td>18.70</td>
<td>25</td>
<td>30</td>
<td>3</td>
<td>70,800</td>
<td>0.48</td>
<td>0.034</td>
<td>6.2</td>
<td>0.07</td>
</tr>
<tr>
<td>Dried</td>
<td>18.70</td>
<td>25</td>
<td>30</td>
<td>6</td>
<td>+111,000</td>
<td>0.47</td>
<td>0.036</td>
<td>6.4</td>
<td>0.08</td>
</tr>
<tr>
<td>Dried</td>
<td>19.12</td>
<td>30</td>
<td>35</td>
<td>3</td>
<td>228,000</td>
<td>0.51</td>
<td>0.039</td>
<td>6.9</td>
<td>0.08</td>
</tr>
<tr>
<td>Dried</td>
<td>19.53</td>
<td>25</td>
<td>30</td>
<td>3</td>
<td>57,000</td>
<td>0.41</td>
<td>0.032</td>
<td>7.3</td>
<td>0.08</td>
</tr>
</tbody>
</table>

* previously subjected to freeze-thaw cycles  + additional number of cycles

As shown in Figure 5.25 the deformation behaviour can be divided into an elastic and a permanent deformation. Figure 5.26 gives an example of the development of the permanent deformation during a particular test.

![Figure 5.25](image)

**Figure 5.25**

Principle of stress-strain behaviour during the uniaxial cyclic loading test

![Figure 5.26](image)

**Figure 5.26** - Development of permanent deformation in uniaxial cyclic loading test

With respect to the results indicated in Figure 5.26, it is noted that a certain amount of permanent deformation has developed during the repeated load phases when compared with the first static phase. Even though the dynamic loads contribute to the development of permanent deformation in EPS, the deformation levels stay low, even when a rather
high cyclic load is applied. Also a large number of load repetitions (up to 270,000 cycles) did not result in an increase of the elastic deformations.

### 5.6.2.3 Remarks on the introducing tests on EPS20

The dynamic elasticity modulus $E_{\text{dyn}}$ of EPS20 calculated from the dynamic cyclic stresses and strains, showed values between 6.2 and 8.3 MPa in case of the maximum expected stresses ($\pm 35$ kPa) in the EPS sub-base. The values of Poisson’s ratio $\nu$, calculated from cyclic radial and vertical strains, varied between 0.07 and 0.11. No difference in the material behaviour was observed as an effect of the various sample states (dry, wet and dried) or the load frequency (3 and 6 Hz respectively).

The maximum dynamic loads expected in an EPS sub-base can contribute to the development of a limited amount of permanent deformation in EPS. However, even a large number of load repetitions of a high cyclic ($\sigma_c = 35$ kPa) load did not result in an increase of the instantaneous deformations. This suggests that excessive permanent deformation is not likely to occur in EPS20 in various states under the maximum stresses expected in the sub-base.

### 5.6.3 Uniaxial cyclic loading tests on EPS15 and EPS20 (continued)

The main shortcoming of the previous cyclic load testing was the inaccurate measuring of the total strain values under static and cyclic stresses. Furthermore, the specimens tested under similar stresses were not selected in a consequent way to represent the variety in density for this material. In subsection 5.7.1 the effects of the variation in density are proved to be of significant influence on the EPS material behaviour. Finally, only EPS20 was tested regardless of the fact that other EPS types are also used in pavement structures.

The uniaxial cyclic loading tests to be discussed in this subsection differed from the foregoing tests with respect to all three above mentioned aspects.

a) The vertical displacement transducers were placed directly on the upper loading plate making it possible to determine accurately the zero position before loading and, consequently, measuring of the total strain during the tests.

b) Each test was carried out on three selected specimens taken from different positions inside an EPS block. In such a way the specimen densities reflected the existing variations in reality and the influence of this variety on the test results was minimized.

c) Both EPS15 and EPS20 specimens were tested providing an insight in the stress-strain behaviour of these two most widely used EPS types for road sub-bases.
5.6.3.1 Test specifications

The apparatus used for the tests considered in this subsection was shown earlier in paragraph 5.4.1.1 in Figure 5.5. The set-ups were similar. Only this time more accurate, small range transducers were applied because smaller deformations than those obtained during the compression test were expected.

As noted in the introduction the specimens were chosen to represent the density variations within an EPS block. It was realized by forming sets of three EPS samples, a relatively 'light' one, an 'average' one and a 'heavy' one, taken from different block positions. The categories 'light', 'average' and 'heavy' characterized the sample density with respect to the average block density. The conclusions are based on the averaged test results per sample set. Both EPS15 and EPS20 cylinders were 300 mm high with a diameter of 150 mm.

EPS20 was subjected to repeated loading only in the most critical dry state according to the test results on small samples described in subsection 5.6.2, while EPS15 was tested in dry and wet conditions. By subjecting both wet and dry EPS15 samples to repeated loading the influence of water absorption, if any, on the stress-strain behaviour of this EPS type was tested. Average water absorption among the EPS15 specimens amounted 1.56% v/v. Details over the preparation of the EPS15 samples are given in Section 5.3.

The tests were carried out force-controlled with the total loading consisting of two components, i.e. a static and a cyclic one. The static stress component was equal to 15 kPa for all performed tests. This stress value represented the dead weight of the upper pavement layers laid above the EPS sub-base in a pavement structure with average layer thicknesses. The cyclic stress component varied between 10 and 50 kPa for EPS15, and between 15 and 75 kPa for EPS20. To select these stress levels the results of the compression tests on EPS15 and EPS20, described in subsection 5.6.1, were used. The cyclic stress values were chosen to make investigations of the EPS material behaviour within the elastic region as well as when plastic deformations possibly occur.

The number of applied loading cycles per test was selected to be high enough to ensure that permanent deformation was obtained under the considered stress level. Accordingly, 100,000 loading cycles were applied during each test in the series. The loading frequency was selected to be 4 Hz representative to the loading frequency in the EPS sub-base due to a passing vehicle above (see Section 3.3). The number of load repetitions in combination with the frequency of 4 Hz plus two short periods, at the beginning and at the end of each experiment when only static stress was applied, resulted in a total loading time of approximately 8 hours per test. The shape of the stress signal was a haversine.

5.6.3.2 Strain development under cyclic loading

Strain values measured on dry EPS15 cylinders under a static stress of 15 kPa combined with a cyclic stress of 10, 20, 35 or 50 kPa, are illustrated in Figure 5.27. From the figure
it can be seen that no significant permanent deformation occurred for cyclic stress values lower or equal to 20 kPa. The cyclic strain component under a 10 kPa cyclic stress amounted 0.17% while the corresponding total strain value \((e_{\text{tot}} - e_{\text{st}} + e_{c})\) was approximately 0.5%. In case of a cyclic load component of 20 kPa the cyclic strain was equal to 0.34% and the total strain to 0.70%.

![Diagram](image)

**Figure 5.27 - Development of strains in dry EPS15 samples under combined static and cyclic stresses**

A cyclic stress of 35 kPa led to development of significant permanent deformations, while a 50 kPa cyclic stress resulted in large permanent deformations of the loaded dry EPS15 cylinders. For a 35 kPa cyclic stress the cyclic strain component ranged between 0.77% (after a few load repetitions) and 0.97% (after 100,000 cycles) while the corresponding total strain values were 1.15% and 2.28% respectively. The illustrated curve for \(\sigma_c = 50\text{ kPa}\) (with *) regards the only EPS15 specimen where the measured deformations stayed within the range of the displacement transducers.

The strain values found on wet (actually either wet or exposed to freeze-thaw cycles) EPS15 cylinders under combined static and cyclic loading are shown in Figure 5.28. As in the case of dry EPS15, no significant permanent deformation occurred under cyclic stress lower or equal to 20 kPa. The cyclic strain values amounted to 0.16% (\(\sigma_c = 10\text{ kPa}\)) and 0.37% (\(\sigma_c = 20\text{ kPa}\)) while the total strain values were 0.45% and 0.70% respectively.
Somewhat lower permanent deformations than for dry specimens were measured under 35 kPa cyclic loading. During the cyclic loading the cyclic strain value increased from 0.74% to 0.84% and the total strain from 1.11% to 1.47%. Large permanent deformations occurred in wet EPS15 specimens under a cyclic stress of 50 kPa. Here again, the presented curve for the highest stress level (the curve with *) does not represent the development of the average strain but the resulting strain values belonging to the only specimen where deformations stayed within the range of the used displacement transducers.

By comparing the graphs in Figures 5.27 and 5.28 it can be seen that water absorption of EPS15 did not negatively affect the material behaviour. Practically, under a cyclic stress of 10 kPa, the same deformations occurred on dry and wet EPS15 cylinders, slightly higher strains were measured on dry samples under 20 kPa, while the difference increased when dry and wet EPS15 cylinders were exposed to a cyclic stress of 35 kPa or higher.

Development of the strain values found on dry EPS20 specimens under cyclic loading (for four different stress levels) are presented in Figure 5.29. After 100,000 load repetitions neither a cyclic stress of 15 kPa nor 30 kPa combined with a static stress of 15 kPa, caused any significant permanent strain. Measured cyclic strains had constant values of 0.17% (σ_c = 15 kPa) and 0.34% (σ_c = 30 kPa), while accompanying total strains were 0.42% and 0.60% respectively.
Significant vertical permanent deformations were obtained when the cyclic stress component was increased to 50 kPa. The cyclic strain value was 0.62% at the beginning of cyclic loading increasing up to 0.72% at the end while accompanying total strain developed from 0.9% to 1.3%. Further increase of the cyclic loading to 75 kPa led to large permanent deformations in the tested EPS20 cylinders manifested as an increasing gradient of the permanent strain-loading repetitions curve (°).

5.6.3.3 Dynamic modulus of EPS15 and EPS20

The values of EPS15 dynamic modulus of elasticity found for dry material conditions are shown in Figure 5.30. For cyclic stress values of 10 kPa and 20 kPa the \( E_{\text{dy}} \)-values remained constant under cyclic loading and amounted to 6.1 and 5.9 MPa respectively. Under a cyclic stress of 35 kPa the \( E_{\text{dy}} \)-value decreased from 4.7 MPa (after a few load repetitions) to 3.9 MPa (after 100,000 loading cycles) and permanent deformations increase in the EPS15 specimens. The \( E_{\text{dy}} \)-time curve for a cyclic stress of 50 kPa is based on the results obtained in the case of the only tested specimen where vertical deformations did not exceed the transducers range.
Development of the $E_{\text{dyn}}$-values found for wet EPS15 cylinders is illustrated in Figure 5.31. Here again, the dynamic modulus of elasticity stayed constant under cyclic stress levels of 10 kPa and 20 kPa. The $E_{\text{dyn}}$-values were equal to 6.3 MPa and 5.5 MPa respectively. Under a cyclic stress of 35 kPa the dynamic modulus decreased from 5.0 MPa to 4.3 MPa. Similar to the case of dry EPS15, two of three wet EPS15 cylinders collapsed before finishing the test when a cyclic stress of 50 kPa was applied so that the $E_{\text{dyn}}$-time curve (°) drawn for this stress level illustrates material behaviour of the best specimen.
By comparing Figures 5.30 and 5.31 the conclusion is being confirmed once again that different material conditions do not affect EPS15 material behaviour under combined static and cyclic stresses.

The dynamic modulus values for EPS20 in dry condition are graphically presented in Figure 5.32. The $E_{\text{dyn}}$-values were equal to approximately 9.3 MPa and 9.0 MPa under cyclic stresses of 15 kPa and 30 kPa respectively. A cyclic stress of 50 kPa caused permanent deformations in EPS20 specimens and the $E_{\text{dyn}}$-value decreased from 8.3 MPa, at the beginning, to 7.6 MPa at the end of the test. Under a cyclic stress of 75 kPa large permanent deformations occurred in the EPS20 cylinders (the curve with *).

![Uniaxial cyclic loading test on EPS20 (dry)](image)

Figure 5.32 - Evolution of dynamic modulus of elasticity of dry EPS20 samples under combined static and cyclic stresses

5.6.4 Cyclic loading tests on EPS20 at low temperatures

Generally speaking plastic materials should show higher resistance against deformation in compression with decreasing temperatures [12, 13]. The results discussed in subsection 5.4.2 confirmed that low temperature between -8.6 and -12.9°C has no negative impact on EPS20 stress-strain behaviour in compression. In this test series it was investigated whether cyclic loading at low test temperatures, in combination with water absorption and exposure to freeze-thaw cycles in the preparation stage, effects EPS20 material behaviour negatively, or might even lead to damage in case stresses are applied which are representative for sub-base conditions.
5.6.4.1 Test specifications

The stress-strain behaviour at low temperature of EPS20 was determined under cyclic loading with a loading speed between 4 and 14 mm/min, corresponding to a strain rate of 2 and 7%/min respectively. Under "loading speed" is understood the speed at which the electric motor drives the crosshead which loads a sample. The tests were performed under strain-controlled conditions. The applied strain rates of 2 and 7%/min, which were dictated by the device limits, did not result in realistic loading frequencies mentioned in Chapter 3. However, applying more sophisticated equipment was impossible, because it was judged too risky to use the advanced servo-hydraulic equipment at a test temperature lower than T=-12°C.

In the scope of the considered cyclic tests the samples were loaded either by only cyclic stresses or both static and cyclic stresses. $\sigma_s$ varied from 0 up to 25.5 kPa. The cyclic stress component $\sigma_c$ had values between 30.6 and 56.0 kPa. The intention was that the total vertical stress is always approximately equal to the total representative maximum vertical stress of 55 kPa. The number of cyclic compression loadings per test varied from few hundred to a few thousand (for higher strain rate).

Figure 5.34 shows the applied stresses versus time. The stresses during the compression tests were observed indirectly through measurements of the force by means of a force meter mounted under the plate with the sample. Values of the deformation were determined from the set loading speed, i.e. strain rate, and the duration of the test.

The compressed EPS20 specimens had the same cylindrical shape with a 100 mm diameter and a height of 200 mm. Three sets of specimens were used: dry, partly saturated with water (wet) and exposed to freeze-thaw cycles. The wet specimens were immersed under water during one year (see subsection 5.3.2), and contained on average 1.56 vol.% water. The water absorption degree of the specimens treated by freeze-thaw cycles (see subsection 5.3.4) amounted up to 1.1 vol.%.

5.6.4.2 Resulting $E_{\text{EPS20}}$ modulus

The resulting E-moduli can be classified with respect to the sample state (dry, wet, and freeze-thaw treated), to the loading speed (4 and 14 mm/min, equal to 2%/min and 7%/min respectively), to the testing temperature (normal and low temperature) and to
the applied stress components (only \( \sigma_x \) or \( \sigma_y \) + \( \sigma_z \)). The measured E-values in all tests varied between 6.9 and 14.1 MPa. All the results are listed in [5].

The E-value of dry samples varied between 7.0 and 9.2 MPa. Wet sample E-values varied between 9.9 and 14.1 MPa. Finally, the samples treated by freeze-thaw cycles had E moduli from 7.0 up to 10.0 MPa. The samples with higher water absorption have thus larger values for the modulus of elasticity.

5.6.4.3 Concluding remarks on EPS20 behaviour at low temperatures

Compression tests carried out at temperatures below 0°C resulted in E-moduli of 7.0 up to 14.1 MPa. When EPS was in a wet state or previously exposed to freeze-thaw cycles, no decrease in the \( E_{EPS} \) value was observed. On the contrary, the wet samples all had a higher modulus of elasticity than the dry samples. The explanation for this could be the higher thermal conductivity of the wet specimens which therefore cool down faster, combined with the fact that polystyrene shows higher resistance against deformation in compression with decreasing temperature [12]. All in all, it seems very likely that the mechanical behaviour of EPS is not significantly affected by freezing or by water absorption or by the combination of these.

5.6.5 Concluding remarks on uniaxial cyclic loading tests

The values of Poisson’s ratio \( \nu \), calculated from cyclic radial and vertical strains, varied between 0.07 and 0.11. No difference in the material behaviour was observed as an effect of the various sample states (dry, wet and dried) or the load frequency (3 and 6 Hz respectively).

The maximum dynamic loads expected in an EPS sub-base can contribute to the development of a small amount of permanent deformation in EPS20. Furthermore it was observed that even a large number of load repetitions of the considered high cyclic load (max \( \sigma_c < 35 \text{ kPa} \)) did not result in an increase of the elastic deformations. This indicates that no damage has been induced in the specimens and it suggests that excessive permanent deformation is not likely to occur in EPS20 in various states under the maximum stresses expected in the sub-base.

Under a static stress of 15 kPa combined with a cyclic stress lower or equal to 20 kPa EPS15 seems to behave elastically. Accompanying elastic cyclic strain values are about 0.37% while the total strain value stays within 0.7%. The considered strain values represent average values for a whole EPS15 block including the effects of typical density variations existing within an EPS block. Elastic behaviour of EPS15 under the above mentioned stress values is independent of material condition, i.e. water absorption or previous exposure to freeze-thaw cycles.
Corresponding dynamic modulus of elasticity of EPS15 blocks stays constant with a minimum $E_{\text{dyn}}$-value of 5.5 MPa under combined stresses $\sigma_c = 15$ kPa and $\sigma_c \leq 20$ kPa.

Permanent deformation will develop in the EPS15 blocks built-in in a pavement structure if a cyclic stress component caused by traffic load has values equal to 35 kPa or more. Dead weight of the upper pavement layers is assumed to result in a static stress of 15 kPa at the top of the EPS15 sub-base.

The resistance to permanent deformation of the blocks of EPS20 is significantly higher than that of EPS15. A cyclic stress of 30 kPa combined with a static stress of 15 kPa does not cause any significant permanent strain. Corresponding cyclic strain has a constant value of 0.34% while accompanying total strain amounts 0.6%, which means a stiffer stress-strain behaviour of EPS20 under $\sigma_c = 20$ kPa compared to EPS15 under $\sigma_c = 20$ kPa.

The $E_{\text{dyn}}$-value of EPS20 blocks under considered stress conditions is equal to approximately 9.0 MPa.

Significant vertical permanent deformation occurs in EPS20 blocks exposed to a 15 kPa static stress, when the cyclic stress component amounts 50 kPa or more.

The dynamic modulus of elasticity of EPS20 does not decrease at low temperatures. Even if EPS is in a wet state or if it was previously exposed to freeze-thaw cycles, no decrease in the $E_{\text{EPS}}$ value was observed. From the experimental results it can be concluded that the behaviour of EPS under cyclic loading is not significantly affected by freezing or by water absorption or by the combination of these.

5.7 DYNAMIC MODULUS DETERMINED BY ELECTRO-DYNAMIC METHOD

5.7.1 Introduction

Though EPS foam has been used in pavement structures for a few decades no applicable method is developed for quality control of the EPS blocks on site. In this section two electro-dynamic methods, the ultrasonic test and the method based on fundamental frequency, are investigated for such a purpose. By means of a electro-dynamic method, the elastic modulus at various locations of EPS blocks can be determined and, because the density and elastic modulus of EPS are closely related, the density variation within a block as well. The ultrasonic test method has been proved to be very useful for the testing of concrete quality on construction site.

Loading speed influences the proportionality constant between the elastic strain and the applied stress because of the thermoelastic effect [14]. If in the elastic region a stress is applied to a sample so rapidly that the maximum stress is reached before the sample can exchange any thermal energy with its surroundings, then this is called an adiabatic loading. The heat transfer from the sample is zero and the mechanical work done on the material causes the change in internal energy. Therefore, in the case of adiabatic compression, the temperature of the sample rises. However, this temperature change is usually
small. If, in contrast with the mentioned adiabatic process, a sample is compressed at a sufficiently low speed, then thermal equilibrium between the sample and its surrounding can be maintained (isothermal process). The modulus of elasticity has a higher value (see Fig. 5.35) if the sample is compressed rapidly (adiabatic process) than in the case of a very slowly increasing load (isothermal process).

Determination of the dynamic modulus of elasticity \( E_{\text{dyn}} \) of EPS20 by means of the electro-dynamic method is based on the relation between the propagation velocity, \( v \), of a wave on one hand and the dynamic modulus of elasticity and density of the elastic material on the other hand. This relation can be rewritten in an expression for \( E_{\text{dyn}} \) as a function of the density and the velocity.

\[
v = \sqrt{\frac{E_{\text{dyn}}}{\rho}} \rightarrow E_{\text{dyn}} = \rho v^2 \tag{eq. 5.3}
\]

where:
- \( v \) - velocity of sound in a material [m/s]
- \( E_{\text{dyn}} \) - dynamic modulus of elasticity [MPa]
- \( \rho \) - material density [kg/m³]

Relation [eq. 5.3] is valid for the transmission of a longitudinal wave through a rectangular beam. In the next two sections experiments will be discussed, in which the dynamic modulus of EPS20 was determined using this relation.

5.7.2 \( E_{\text{dyn}} \) by means of ultrasonic testing

The velocity of wave propagation through a rectangular beam can be determined by measuring the time delay of a wave travelling from one end of the beam to the other. The beam length \( l \) is known. If this length is divided by the measured passing time \( t \), the propagation velocity \( v(=l/t) \) of the wave in the material is found. The material density, \( \rho \), in the relation [eq. 5.3] is easy to find from the beam weight and volume.

The device for carrying out the ultrasonic test is given in Figure 5.36. The apparatus consists of a central unit with a display and a receiver and transmitter of the ultrasonic waves. The transmitter, put on one end of the beam, repeatedly initiates waves which are received by the receiver at the opposite end. The passing time is read on the display with an accuracy of \( 10^6 \) sec (\( \mu s \)). A high wave frequency (in the range of few MHz) enables such a high accuracy in measuring of time.
The tested beams of EPS20 had dimensions either 50×50×300 or 50×50×500 mm. The calculated $E_{\text{dyn}}$ had values between 10.1 and 15.0 MPa. The lower value was found in the case of the longer 500 mm beams. For the shorter 300 mm beams $E_{\text{dyn}}$ had an average value of 14.4 MPa. It points out that the sample length undoubtedly influences the measured $E_{\text{dyn}}$ values. Nevertheless, the results can be considered as an indication for the $E_{\text{dyn}}$ of EPS20.

This experiment was repeated on a block of EPS20. By means of measurements on the different points on the EPS block an attempt was made to determine the influence of the distance from the edge on the $E_{\text{dyn}}$ values which were calculated using the relation [eq. 5.3]. There are two reasons why this distance affects the calculated $E_{\text{dyn}}$ values. The vicinity of the block edge influences the propagation of the wave and the bulk density varied within the block.

Figure 5.37 shows an example taken from the literature [15] of the variation in bulk density between the layers (equal to 8% of the average density of the whole EPS block).

In Figure 5.38 the calculated local dynamic modulus (our measurements) is shown as a function of the distance from the block edges.
In Figure 5.39 the local \( E_{\text{dyn}} \) is shown for the points on the diagonal of the block side. From those figures it can be concluded that up to a distance of 0.15 m the edge has a significant influence on the measured \( E_{\text{dyn}} \) modulus. The \( E_{\text{dyn}} \) values measured close to the edge will be lower than the values measured in the central part of the block.

The ultrasonic test method considered in this section seems to be applicable to quality control of EPS blocks on site. By means of this method, the elastic modulus at various locations of EPS blocks can be determined, since the density and elastic modulus of EPS are closely related. However, more work has to be done in order to validate the results obtained by means of ultrasonic testing because the resultant \( E_{\text{dyn}} \) values are higher than the values obtained by means of other measuring techniques. Secondly, a modification of existing equipments is necessary in such a way that the ultrasonic apparatus can be fully applicable for testing of the large EPS blocks on a construction site.

### 5.7.3 \( E_{\text{dyn}} \) by means of fundamental frequency

Another way to determine \( E_{\text{dyn}} \) using relation [eq. 5.3] is by measuring the resonance frequency. The experiment consists of transmitting a longitudinal wave of variable frequency through the prism and adjusting the frequency of vibration until the prism resonates. The central section of the prism is fixed. A node must occur at this section, and the free ends are antinodes. Thus the wavelength of the vibrations in the prism of the considered material is twice the length of the prism.

If the frequency of the fundamental mode of vibration is \( n \) cycles per second, then the velocity of wave propagation through the prism is given by \( v = n \lambda = 2n l \). Therefore, the relation [eq. 5.3] can be rewritten as follows:

\[
 v = n \lambda = 2l n \Rightarrow E_{\text{dyn}} = 4 n^2 l^2 \rho \quad \text{[eq. 5.4]}
\]

where:
- \( n \) - fundamental frequency [Hz]
- \( \lambda \) - wavelength [m]
- \( l \) - length of the prism [m]
The equipment needed for determination of $E_{\text{dyn}}$ by means of fundamental frequency is shown in Figure 5.40. The experimental set up consists of a variable audio frequency oscillator connected with a vibrator and a vibration detector attached to a measuring device (valve voltmeter). The electro-magnetic vibrator is placed at one end of the prism while the vibration detector is placed at the opposite end.

The experiment was carried out on EPS20 prisms of two different lengths, approximately 300 mm and 500 mm respectively. The values for $E_{\text{dyn}}$ that were obtained from the tests varied from 10.3 to 14.4 MPa. The measured fundamental frequencies ranged between 0.8 kHz and 1.4 kHz for 500 and 300 mm beams respectively. Complete results were reported in [5].

### 5.7.4 Concluding remarks on $E_{\text{dyn}}$ and ultrasonic tests

The values of the dynamic modulus $E_{\text{dyn}}$ found by measuring the velocity of waves propagating through the EPS20 were between 10.1 and 15.0 MPa. The values of $E_{\text{dyn}}$ from the fundamental frequency varied between 10.3 and 14.4 MPa. These two electro-dynamic methods give an insight into the mechanical behaviour of EPS for loading frequencies that are much higher than those generated by traffic. Nevertheless, the above mentioned values of $E_{\text{dyn}}$ indicate the performance of EPS20 in the case of high loading frequencies.

When the dynamic modulus of EPS is measured on an EPS block by means of the electro-dynamic ultrasonic method one has to realize that the $E_{\text{dyn}}$ values measured close to the edge will be lower than the values measured in the central part of the block. However, because density and elastic modulus of EPS are closely related, the ultrasonic test method has potential to be used on site to determine the elastic modulus of various locations on EPS blocks for quality control purposes. More work has to be done in order to validate the results obtained by means of the ultrasonic testing method. Also, a modification of the existing equipments is necessary so that the ultrasonic apparatus can be fully applicable for testing large EPS blocks on construction sites.
5.8 CONCLUDING REMARKS ON MATERIALS RESEARCH

- EPS20 absorbs water very slowly and to a limited extent. The average absorption of water by means of immersion was 1.53% v/v after one year. The asymptotic value of 2% v/v can be assumed as the maximum percentage of water which EPS20 will absorb. This value is much lower than the values given in the literature. It should be emphasized that the rate of water absorption is dependent on the shape of the samples and the preparation (cutting) of the surfaces. A larger contact surface relative to the sample volume increases the rate of absorption even over a larger period.

- The dominant transport mechanism responsible for water absorption seems to be diffusion. Initiated by a vapour concentration difference, vapour penetrates progressively through the thin walls of the cells. Further diffusion into the inner layer cells is more difficult.

- Water absorption can be accelerated by exposing EPS to freeze-thaw cycles. This treatment stimulates capillary condensation of the vapour diffused into the material. At low temperatures, the vapour condenses and accumulates in the cells. Therefore the inner vapour concentration decreases, the vapour concentration gradient into the cells increases, which encourages further diffusion.

- The total vertical permanent strain in the EPS sub-base due to the dead weight of the top layers ($\sigma = 20$ kPa) has a range of approximately 0.5% in the case of long-term loading due to very heavy pavement structures. The immediate vertical strain in EPS blocks, a significant part of the total vertical strain, amounts to approximately 0.3%. Remaining vertical strain is the result of creep.

  Under a static stress of about 20 kPa, corresponding to the dead weight of the pavement top layers, the creep of EPS20 amounts to 0.2% after more than a year. About half of the expected maximum creep in the EPS occurs already within the first day. Therefore, the small additional permanent vertical deformation of the pavement structure caused by creep of the EPS sub-base layer is of minor practical importance for pavement design.

- The linear-elastic region for EPS20 under slow compressive loading at normal and low temperatures characterized by a boundary strain values, $\varepsilon_{bl}$, is in the range of 0.4% up to 0.6%. The corresponding boundary stress values vary between 40 and 50 kPa. Values of their E moduli vary between 6.2 and 12.4 MPa. Beyond the elastic region EPS offers a large plastic strain capacity while the load carrying capacity is maintained.

- After the cell structure is damaged due to overloading, water penetrates faster into the EPS and accumulates not only in small voids between the fused cells but in the collapsed cells as well. This causes an increase of the maximum water content in the EPS when deformed beyond the failure limit of its cell structure.
Under a static stress of 15 kPa combined with a cyclic stress lower or equal to 20 kPa EPS15 seems to behave elastically. Accompanying elastic cyclic strain (average value representative for a whole EPS15 block) amounts to about 0.4% while the total strain value stays within 0.7%. Corresponding dynamic modulus of elasticity of EPS15 blocks stays constant with a minimum $E_{dyn}$-value of 5.5 MPa.

Permanent deformation will develop in the EPS15 blocks built-in in a pavement structure if a cyclic stress component caused by traffic load has values of 35 kPa or higher. The dead weight of the upper pavement layers is assumed to result in a static stress of 15 kPa at the top of the EPS15 sub-base.

The resistance to permanent deformation of the blocks of EPS20 is significantly higher compared to EPS15. A cyclic stress of 30 kPa combined with a static stress of 15 kPa does not cause any permanent strain of significance in EPS20. Corresponding cyclic strain (average value for a whole EPS20 block) has a constant value of 0.34% while accompanying total strain amounts 0.6%, which means a stiffer behaviour of EPS20 under $\sigma_c = 30$ kPa compared to EPS15 under $\sigma_c = 20$ kPa. The $E_{dyn}$-value of EPS20 blocks under considered stress conditions is equal to approximately 9.0 MPa.

An excessive permanent deformation is not likely to occur in EPS20 under the maximum stresses which can be expected in the sub-base. Significant vertical permanent deformation occurs in EPS20 blocks exposed to a 15 kPa static stress, when the cyclic stress component amounts to 50 kPa or more.

The modulus of elasticity of neither EPS20 nor EPS15 decreases at low temperatures. Even when EPS is in a wet state or when previously exposed to freeze-thaw cycles, no decrease in the $E_{EPS}$ value is observed. All in all, low temperatures, water absorption level and exposure to freeze-thaw cycles, separately or combined, seem to have no negative influence on the mechanical behaviour of EPS20.

The Poisson's ratio values of EPS20, calculated from cyclic radial and vertical strains, varied between 0.07 and 0.11.

The values of the dynamic modulus $E_{dyn}$ found by electrodynamic methods were between 10.1 and 15.0 MPa. These EPS20 moduli are representative for loading frequencies much higher than those caused by traffic.

The ultrasonic test method has the potential to be used on site to determine via the elastic modulus the density of various positions in EPS blocks for quality control purposes. This is made possible by the close relationship between the density and the elastic modulus of EPS. More work has to be done in order to validate the results obtained by means of the ultrasonic testing method. Also, the modification of the existing equipment is necessary so that the ultrasonic apparatus can be fully applicable for testing of large EPS blocks on construction sites.
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CHAPTER 6

TEST PAVEMENTS ANALYSIS

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6.1 INTRODUCTION

The use of EPS Geofoam instead of a traditional "heavy" sand sub-base can reduce or even eliminate the additional load on the subsoil of poor load-bearing capacity (peat), thus decrease or eliminate the settlement of the road pavement structure. Such a solution of a geotechnical problem leads, however, to pavement design problems because the EPS is not only a light-weight but also a low-modulus material. Consequently, the application of the EPS blocks in the sub-base effects significantly the load spreading in the pavement layers. The EPS sub-base does not have the function of load spreading, in contrast with the traditional sub-base. The bearing function has to be accomplished by the upper pavement layers. This somewhat specific behaviour was the reason to perform an in-depth structural analysis of pavement structures with an EPS sub-base which is described in this chapter.

The present design guidelines for pavement structures with an EPS sub-base are still mainly empirical. A standard worldwide accepted design procedure does not exist. Pavement structures with an EPS sub-base differ most significantly with respect to the roadbase type. In Scandinavia the pavement structures have a gravel base layer over a concrete capping layer, while in The Netherlands the pavement roadbase usually consists of only unbound materials over the EPS blocks.

Optimization of the existing EPS pavement design guidelines and their improvement require full-scale testing of pavement structures with alternative roadbase types above an EPS sub-base on one hand, and the use of sophisticated mechanical pavement models to enable numerical analyses on the other. The measurements on the test sections provide data for the comparison of the tested pavements and validation of the numerical models. The calculated stresses and strains in the sections are also compared with each other as well as with the allowable values. Conclusions drawn from the analyses of test pavements completed with the conclusion obtained from additional numerical analyses of alternative (not tested) pavements contribute to the optimization of the design guidelines for pavement structures with an EPS layer.

To investigate flexible pavement structures with an EPS sub-base, six full-scale pavement test sections were constructed at the Federal Highway Research Institute (BundesAnstalt für Strassenwesen - BASf) at Bergisch Gladbach, Germany. The measurement program included registration of the horizontal tensile strains at the bottom of the asphalt layer, surface deflections, degree of compaction degree in the gravel layer and rut depths during long-term loading tests.

The author could participate in that research program through contacts that were laid by Shell Nederland Chemie. The contribution of the Delft University of Technology to the program consisted of falling weight deflectometer tests on the test pavements, repeated load triaxial tests on the granular base material and extensive numerical analyses using the DIANA program [1]. The DIANA program (DIspacement method ANAlyzer) contains a non-linear elastic material model suitable for simulation of the stress-depen-
dent behaviour of unbound gravel materials [2]. By using the axial symmetric model the stress, strain and displacement values in the asphalt pavements with the EPS sub-base were calculated.

The test sections to be analyzed, built at Federal Highway Research Institute (BASf) at Bergisch Gladbach, Germany, are described in section 6.2. Three different types of pavement structures, with respect to the roadbase layers, were constructed above a 1 m thick EPS sub-base. The asphaltic concrete toplayer had two thicknesses, 140 and 220 mm respectively. Therefore six test sections were considered in total.

In section 6.3 the measuring program performed on the test sections is described as well as the measuring results regarding horizontal tensile strains at the bottom of the asphaltic concrete layer, the cement concrete layer and the cement-treated capping layer, the degree of compaction in the gravel layer, truck serviceability on unbound gravel layers and rut depths during long-term loading tests. Also details about measured surface deflection bowls and the back-calculation procedure for determination of the elastic layer moduli are presented.

Section 6.4 describes the background of the modelling and the calculations as well as the results of calculations on flexible pavement structures with an EPS sub-base using the DIANA finite elements program. The design of the finite element mesh is described and the chosen form and the dimensions are discussed in subsection 6.4.2 while the applied material models and modelling of the loading are explained in 6.4.3 and 6.4.4 respectively. Subsection 6.4.5 deals with the results obtained in three calculation sessions. Firstly, pavement structures with an EPS or a sand sub-base were analyzed and compared with each other. Secondly, the pavement structures with an unbound roadbase and with a load spreading capping layer above the EPS sub-base were analyzed. In the third calculation session the characteristics of the EPS block structure were taken into account and the influence of the position of the joints between the EPS blocks on stresses, strains and deformations was investigated. It should be noted that the results of these calculations should be treated with care since the pavement model in that case is far from reality. The axi-symmetric analyses implied that rings of EPS were analyzed instead of the true block structure. A much more detailed analysis on the effect of joints in the block structure on the pavement behaviour is given in chapter 9 where a true 3-D analysis is made using the CAPA program.

The verification of the numerical axial symmetric analysis of the test sections at BASf is described in section 6.5. The parameters of the non-linear material model for the unbound roadbase material were determined by means of cyclic loading triaxial tests. The other materials and the subsoil were assumed to behave linear elastically and their moduli of elasticity were obtained by means of a back-calculation procedure using the results of falling weight deflection measurements. To verify the DIANA models the measured horizontal strains at the bottom of the asphalt layer and the measured deflections were used. The influence of a load-distributing intermediate layer on top of the EPS sub-base on the unbound roadbase material was analyzed by means of the calculated stresses and strains. Finally, concluding remarks are given.
6.2 ASPHALT TEST SECTIONS AT BAST

To investigate flexible pavement structures with an EPS sub-base, six full-scale test sections with dimensions of 13.0 \times 3.7 \text{ m}, alternatively 12.0 \times 3.7 \text{ m} each were constructed at the BAST at Bergisch Gladbach, Germany. The pavement structures of the test sections differ with respect to the roadbase type and the asphalt thickness.

6.2.1 Construction of test sections

The test pavement structures were built at BAST inside a hall with a concrete pit, some 3.5 \text{ m} deep. The test pit consisted of reinforced concrete with a high percentage of reinforcement. For calculation purposes it could therefore be considered as a rigid bottom. To examine the various issues, this pit was filled with water-sensitive soil (loam) and a high "groundwater" level was simulated. This subsoil was covered with a layer of approx. 100 \text{ mm} sand for an easier placement of the EPS blocks. The EPS layer was realized by two layers of blocks, resulting in a 1 \text{ m} thick embankment with a density of only 20 \text{ kg/m}^3 (EPS20). Above the 1 \text{ m} EPS sub-base three different types of pavement structures were constructed.

The pavement structures on top of the EPS sub-base differed from each other with respect to the roadbase. Generally speaking all existing pavement structures with an EPS layer can be classified into two main types with respect to the roadbase layers. The first type of pavement structures, originally used in Norway, has a load-spreading capping layer of reinforced concrete on the top of an EPS sub-base (marked as 'CB' type in Figures 6.1 and 6.2). The second type has a roadbase which consists exclusively of unbound granular materials ('GB' type). Such pavement structures are widely used in the Netherlands. As an intermediate case between the 'CB' and 'GB' types, the third structure with a layer of 200 \text{ mm} of cement-stabilized sand placed above the EPS sub-base (marked as 'SB' type in Figures 6.1 and 6.2) was constructed and tested at BAST.

Two asphalt layer thicknesses were applied. One half of the test area was covered with a 220 \text{ mm} thick asphalt layer and the other with a 140 \text{ mm} thick layer. The asphalt thicknesses correspond to the requirements for two traffic classes (II and VI) of the German pavement design standards. Owing to the fact that the test pit was located in a building, the boundary conditions for the different tests could be maintained constant. This mainly applied to the groundwater level in the test pit and the pavement surface temperature which was kept at an average value of 20^\circ \text{C} during all the tests.
The different pavement structures with an equal asphalt thickness were placed in succession along the axis of the testing hall, the sections of different asphalt thickness next to each other as shown in Figure 6.1.

6.2.2 Layer thicknesses and materials

Details of the layer structures of the test sections 'CB', 'GB' and 'SB' are presented in Figure 6.2. The total thickness of the layers above the EPS sub-base was 560 mm. Differences in asphalt layer thicknesses have been compensated by the thickness of the gravel layers.

The first structure ('CB') had a 140 mm thick lightly reinforced concrete layer placed directly over the EPS layer (test sections 'CB'1 and 'CB'2 in Figures 6.1 and 6.2). The concrete layer was overlaid with an unbound gravel layer. Prior to construction, all the materials had been tested in the laboratory in order to be able to compare test and computation results with the conditions found in practice. From the concrete layer, test specimens had been taken and subjected to bending tests until failure. An average modulus of elasticity of 26,000 MPa was determined.

The second 'GB' structure corresponded to a construction method used in the Netherlands: no load spreading capping layer was used, only a thin plastic sheet was placed on the EPS layer which was then covered directly by an unbound gravel layer (test sections 'GB'3 and 'GB'4 in Figures 6.1 and 6.2). Due to the plastic sheet, the joints between the EPS blocks would remain free from contamination by loose base material.
The third 'SB' structure represented an intermediate solution to those described above (test sections 'SB'5 and 'SB'6 in Figures 6.1 and 6.2). In this case, a layer of 200 mm non-reinforced cement-stabilized sand was placed on the top of the EPS sub-base. The use of this method was guided by the idea that sand as a low-cost construction material will always be available in sufficient quantities at locations with poor subsoil conditions. An average modulus of elasticity of 11,000 MPa was determined for the cement stabilized sand layer. This layer was also covered by a layer of unbound gravel.

### 6.2.3 EPS block patterns

The EPS blocks with dimensions 0.5×1.0×4 m and 0.5×1.25×2.50 m were placed by hand according to the pattern illustrated in Figure 6.3 [3]. The patterns of the two EPS layers were rotated for 180° with respect to each other. When placing the blocks for each of the pavement sections, attention had to be paid to avoid the creation of joints directly above each other. The two layers of blocks were not anchored or jointed in any way. The blocks were cut using a hot wire.
6.3 MEASUREMENTS ON TEST SECTIONS

The measuring program included measurements of the horizontal tensile strains at the bottom of the asphalt layer, the cement concrete layer and the cement-treated capping layer, surface deflections, compaction degree in the gravel layer and rut depths during long-term loading tests. The strain and compaction measurements described in this section 6.3 as well as long-term tests were performed by the BASt staff [4, 5], while the measurements of the surface deflections, the back-calculation analysis and the cyclic loading triaxial tests were carried out by the staff of the Laboratory for Road and Railroad Research of the Delft University of Technology. The strains at the bottom of the pavement layers were monitored by means of the built-in strain transducers. To measure surface deflections the falling weight deflectometer (FWD) was utilized.

6.3.1 Measurements on gravel layer

In order to realize maximum compaction of the gravel layer, three devices were tested: a vibrating roller (1 t), a static roller (1 t) and a plate vibrator (320 kg). The best compaction results came from the plate vibrator. The vibrating roller proved to be unsuitable. Proctor tests were performed to determine optimal water content and the dry density. Compaction in situ was monitored continuously by means of a nuclear probe. The final compaction state was reached when no further increase in density was obtained from further compacting passes.

The results of the density measurements performed at 20 points within the various sections, showed that compaction in the sections 'CB'1 and 'CB'2 was particularly difficult. Altogether, it was only possible to obtain 100% Proctor density ($\gamma_{p_{r}} = 1,896$ kg/m³) in the upper 0.2 m. No improvement was obtained by further compaction passes. This was believed to be due to resonant vibrations of the cement concrete capping layer caused by the compaction equipment. This resonance vibration could result in loosening of the packing of the granular skeleton. The relative compaction obtained in the upper 0.3 m of the 'GB' structures was $\gamma_{p_{r}} = 104 \pm 1.7\%$; this result is quite good. In the sections 'SB'5 and 'SB'6, $\gamma_{p_{r}} = 102 \pm 2.5\%$ was obtained. This is a result that can be regarded as satisfying.

The deformation modulus was determined using the plate bearing test. The deformation modulus $E_{v}$ was obtained from the deflection under a circular plate, $E_{v1}$ followed from the first load, $E_{v2}$ from the second load in the test. According to the German pavement design standards the second load has to be used to assess the bearing capacity. In the sections with a load distributing layer, the minimum requirements according to the German guidelines, $E_{v2} = 120$ MPa, were either reached or exceeded. In the sections with only the gravel base layer, however, this was by no means the case. There a value of 28 MPa was obtained; such a low value can only occur in poor subsoil conditions which in Germany would call for soil improvement, stabilization or replacement of the existing
material. This would be considered necessary since such a low bearing capacity in the base would mean that the compaction of the pavement layers above was impaired. However, this apprehension was not confirmed. Also adequate compaction of the asphalt layers of these sections was obtained. The measurement results on the gravel layer are summarized in Table 6.1.

<table>
<thead>
<tr>
<th>test section</th>
<th>'CB'1</th>
<th>'CB'2</th>
<th>'GB'3</th>
<th>'GB'4</th>
<th>'SB'5</th>
<th>'SB'6</th>
</tr>
</thead>
<tbody>
<tr>
<td>gravel layer thickness [mm]</td>
<td>200</td>
<td>280</td>
<td>340</td>
<td>420</td>
<td>140</td>
<td>220</td>
</tr>
<tr>
<td>dry density [kg/m³]</td>
<td>1,895</td>
<td></td>
<td>1,967</td>
<td></td>
<td>1,936</td>
<td></td>
</tr>
<tr>
<td>compact. degree Dₚᵣ [%]</td>
<td>100 ± 1.7</td>
<td></td>
<td>104 ± 1.7</td>
<td></td>
<td>102 ± 2.5</td>
<td></td>
</tr>
<tr>
<td>deformation Eᵥ₁ [MPa]</td>
<td>35</td>
<td></td>
<td>14</td>
<td></td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>modulus Eᵥ₂ [MPa]</td>
<td>136 ± 24</td>
<td></td>
<td>28 ± 2</td>
<td></td>
<td>180 ± 39</td>
<td></td>
</tr>
<tr>
<td>truck serviceability ¹ on unbound layers</td>
<td>=</td>
<td>+</td>
<td>x</td>
<td>x</td>
<td>=</td>
<td>=</td>
</tr>
</tbody>
</table>

¹+: well suited =: conditionally x: not suited

Table 6.1 - Measurement results on gravel layer

To test the usability of the unbound layers, tests simulating a 100 kN axle load were carried out by means of a loading plate. The resulting strains were measured at the bottom of the concrete layer and the cement stabilized sand layer. The measured values were compared with the failure strain measured in bending tests. This analysis showed that section 'CB'2 could be loaded by construction traffic without damaging the concrete slab. Where the gravel layer was lowered by 80 mm (section 'CB'1), the strain in the concrete came very close to the failure limit. In this case, only lower wheel loads should be allowed.

The static loading stress in the sections 'GB'3 and 'GB'4 led to very high plastic deformations in the case of heavy loads. From this it may be inferred that these sections should under no circumstances be subject to heavy-vehicle traffic. Only vehicles having a good load distribution might be allowed. The 'SB' structure was only conditionally trafficable. Here, the strains occurring were close to the failure limit, some values were well above that limit.
6.3.2 Measurements on asphaltic concrete layers

6.3.2.1 Horizontal strain due to truck passages

The asphalt strains due to a truck passage were measured. The measurements were taken at two truck speeds with rear wheel loads of 50 kN. For the 'CB' and 'SB' structures, the strains were in the range of values which usually occur on conventional roads (150 to 200 μm). For the 'GB' structure, the strains were very much higher as can be seen in Table 6.2. It was remarkable that the thickness of the asphalt layers had no great influence on the level of the strains.

<table>
<thead>
<tr>
<th>test section</th>
<th>'CB'1</th>
<th>'CB'2</th>
<th>'GB'3</th>
<th>'GB'4</th>
<th>'SB'5</th>
<th>'SB'6</th>
</tr>
</thead>
<tbody>
<tr>
<td>asphalt thickness</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>[mm]</td>
<td>220</td>
<td>140</td>
<td>220</td>
<td>140</td>
<td>220</td>
<td>140</td>
</tr>
<tr>
<td>FWD deflect. (50 kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>[μm]</td>
<td>461</td>
<td>571</td>
<td>847</td>
<td>1711</td>
<td>421</td>
<td>526</td>
</tr>
<tr>
<td>horizon. v=3 km/h</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>[10^6]</td>
<td>125</td>
<td>180</td>
<td>280</td>
<td>280</td>
<td>-</td>
<td>185</td>
</tr>
<tr>
<td>strain v=10 km/h</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>[10^6]</td>
<td>120</td>
<td>160</td>
<td>240</td>
<td>240</td>
<td>-</td>
<td>160</td>
</tr>
</tbody>
</table>

Table 6.2 - Measurement results on asphalt layer

6.3.2.2 Long term loading tests

Repeated loading was performed by means of repeated plate loading tests. Special about these repeated loading was that the plate did not stay at the same location but moved about 1 mm forward after each load pulse. To simulate the lateral wheel load distribution, pulse loads were applied on several tracks. The purpose of this test was the determination of the long term behaviour of pavement structures in terms of permanent deformation (rut depth) development as a function of the number of simulated wheel passes.

A complete test consisted of 10^6 simulated wheel passes. In Fig. 6.4 the results of these tests are summarized. In the case of the test section 'GB'4, the test had to be stopped after approx. 450,000 wheel passes due to the development of surface cracking. The resulting rut reached a depth of 47 mm. The results obtained for the remaining structures, however, showed a resistance to deformation rated as good to very good.
6.3.3 FWD deflection measurements

On all test pavements the surface deflections were measured at three locations by means of a falling weight deflectometer (FWD). The average deflection was taken as the representative value. The deflections under the loading plate centre due to a force of 50 kN are shown in Table 6.2. For the pavements with a load-distributing layer above an EPS sub-base these values were about the same (the difference was approximately 10%). The measured deformation on the 'GB' pavement with a thicker asphalt layer was about twice as large as the deflection measured on the corresponding 'SB' and 'CB' sections. This ratio amounted to about three for the 'GB' pavement with an asphalt layer of 140 mm. In order to be able to determine any stress dependent behaviour, the measurements were performed at three load levels.

6.3.3.1 Back-calculation analysis

The Young's moduli of the test pavement layer materials were determined by back-calculations from measured deflection bowls. To enable the back-calculation, the analyzed pavement structures were simplified. The number of model layers was limited to 5. Poisson's ratio was taken as 0.35 for all materials except for EPS where it was taken as 0.10. Furthermore, the unbound gravel layers were divided into two sublayers with different E-values to simulate the stress dependent behaviour of this material. In those cases where a cement bound layer was placed above the EPS blocks, the EPS layer was considered as part of the subsoil. This means that in those cases a combined modulus for subsoil and EPS layer was obtained. The back-calculations were carried out by means of the linear-elastic multi-layer program BISAR.
### 6.3.3.2 Back-calculated elasticity moduli

The back-calculated layer moduli are presented in Table 6.3. In the following paragraphs the conclusions drawn by summarizing the back-calculation results are listed starting with the deeper laid layers.

<table>
<thead>
<tr>
<th>FWD load</th>
<th>asphalt</th>
<th>gravel</th>
<th>concrete</th>
<th>EPS20</th>
<th>subsoil</th>
</tr>
</thead>
<tbody>
<tr>
<td>[kN]</td>
<td>h=220 mm</td>
<td>h=200 mm</td>
<td>h=140 mm</td>
<td>h=1000 mm</td>
<td></td>
</tr>
<tr>
<td>'CB' 1</td>
<td>35</td>
<td>9000</td>
<td>190</td>
<td>6500</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>9000</td>
<td>240</td>
<td>6700</td>
<td>34</td>
</tr>
<tr>
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<td>70</td>
<td>9000</td>
<td>230</td>
<td>6500</td>
<td>34</td>
</tr>
<tr>
<td>'CB' 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>9000</td>
<td>300</td>
<td>200</td>
<td>4100</td>
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<tr>
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<td>70</td>
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<td>350</td>
<td>180</td>
<td>3600</td>
</tr>
<tr>
<td>'GB' 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>9600</td>
<td>28</td>
<td>18</td>
<td>27</td>
</tr>
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<td>50</td>
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<td>70</td>
<td>7900</td>
<td>28</td>
<td>17</td>
<td>26</td>
</tr>
<tr>
<td>'GB' 4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>2800</td>
<td>88</td>
<td>80</td>
<td>15</td>
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</tr>
<tr>
<td></td>
<td>70</td>
<td>2500</td>
<td>77</td>
<td>77</td>
<td>13</td>
</tr>
<tr>
<td>'SB' 5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>9000</td>
<td>700</td>
<td>600</td>
<td>1300</td>
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</tr>
<tr>
<td></td>
<td>75</td>
<td>9500</td>
<td>450</td>
<td>300</td>
<td>1100</td>
</tr>
<tr>
<td>'SB' 6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>8000</td>
<td>500</td>
<td>400</td>
<td>1600</td>
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<tr>
<td></td>
<td>70</td>
<td>8000</td>
<td>500</td>
<td>400</td>
<td>1500</td>
</tr>
</tbody>
</table>

Table 6.3 - Back-calculated E-modulus values of the test pavement layers based on FWD measurements using a load of 35 kN, 50 kN and 70 kN, respectively.
The E-values back-calculated for EPS20 were found to be between 13 MPa and 34 MPa. Such values are somewhat higher than the modulus of elasticity obtained in experiments [6] or found in literature [7].

The resilient modulus of the unbound gravel base was found to be much lower in the structures without a cement treated capping layer than in structures with such a capping layer.

The calculated Young's modulus for the asphalt ranged between 7,900 and 9,600 MPa (excluding test section 'GB'4), which was lower than the values obtained in the bending tests for corresponding frequency and temperature. This is especially valid for the concrete and cement stabilized sand layers where there was no agreement between the back-calculated moduli and the bending test results.

The low asphalt modulus for section 'GB'4 (structure with a 140 mm asphalt layer above a 420 mm thick gravel roadbase) was because of the fact that cracks already existed in the asphalt. These cracks are most likely due to fatigue because of the high tensile strains that developed during the cyclic plate loading test.

### 6.3.4 Concluding remarks on measurement results

For the BASt pavements the most suitable equipment for compaction of unbound material in pavement structures with an EPS sub-base seemed to be a plate vibrator. The static roller is less efficient than a vibrator, while in this particular case the vibrating roller seems to be not suitable.

Truck serviceability in the construction phase is unconditionally possible only on section 'CB'2 with a concrete capping layer above EPS and a 280 mm unbound roadbase on top of it. Loadings lower than 50 kN per wheel are allowed on roadbases of the sections 'CB'1 (with a reinforced concrete capping layer and a 200 mm thick unbound layer), 'SB'5 and 'SB'6 (with a cement-stabilized sand layer above the EPS sub-base) while the sections 'GB'3 and 'GB'4 (where the roadbase consisted of only unbound gravel material) should under no circumstances be subjected to heavy traffic in the construction phase. It would result in high plastic deformations, and therefore only vehicles with wheel configurations that result in low stresses, e.g. tracklaying vehicles, might be allowed.

The 'GB' pavement structures, as applied at BASt, can bear a low number of standard axle load repetitions before cracking of the asphalt occurs. This is mainly because of the disappointing low modulus value for the asphalt occurs. Investigation into the reason of the very low base modulus of the 'GB' pavement structures is considered to be extremely important since design errors can be overcome if those reasons are known.
6.4 FINITE ELEMENT (FE) AXIAL SYMMETRIC PAVEMENT ANALYSIS

6.4.1 Introduction

Before the test pavements were analyzed calculations were made on pavement structures with an unbound base and with and without a cement treated capping layer using the finite elements (FE) program DIANA. The most important benefit of utilizing the DIANA program was the possibility to model non-linear stress-dependent behaviour in the roadbase layer above the EPS sub-base. These calculations were made to qualify and quantify the effects of an EPS layer on the behaviour of the pavement structure. These results could then be used to explain the behaviour of the BASl test pavements. The pavement structure was simplified to an axial symmetric body. Although this simplification is a rather rigorous one because of the presence of the EPS blocks it was believed that this schematization was sufficiently accurate to analyze the effect of an EPS layer on the behaviour of a pavement structure. The FE mesh is presented in Figure 6.5.

The shortcoming of the model was that the nodes have two degrees of freedom instead of three in the case of a three dimensional model. The node displacement in tangential direction is not possible. Furthermore, only an axial symmetric load can be applied and boundary conditions differ from the real ones. Finally, an adequate analysis of the effects of the EPS block structure on the pavement's behaviour could not be done since only rings instead of blocks could be created.

6.4.2 Axial symmetric FE pavement model

6.4.2.1 Mesh dimensions

The dimensions of the axial symmetric model were a diameter of 10.0 m and a height of 15.0 m. The large model diameter made it possible to neglect the influence of the horizontal boundary conditions on the calculation results. The model height was selected based on BISAR calculations. The supports at the bottom of the model are such that horizontal and vertical displacements are zero, thus simulating a rigid bottom. Using the BISAR model the influence of a rigid bottom on the surface deformations was analyzed.
This influence was negligible when the rigid bottom was located at a depth of 15.0 m or more as shown in Figure 6.6. In literature [8] also DIANA pavement models with smaller dimensions (R = 3.0 m, H = 3.0 m) are mentioned but the larger model with the height of 15.0 m was found to be more suitable [9] and is therefore regularly used in pavement analyses [10, 11].

Using the axial symmetry of the model, the FE mesh was designed for half of its cross section. The stresses, strains and displacements caused by a symmetrical loading are identical on the other side of the axis of symmetry. This area was divided into elements. The mesh was designed with respect to the next considerations:

a) Surface nodes are necessary on the geophone positions of the falling weight deflectometer. The displacements can be calculated only in nodes.

b) In the area with a high stress gradient the elements have to be small. Assuming that the load will be spread under an angle of 45° (in asphalt the load spreading will be better) the area with a high stress gradient can be located in advance. The elements on a greater distance from the load centre are much bigger.

c) The ratio between the vertical and horizontal element dimension can not be too big (≤ 5).

d) The joints between the EPS blocks in a sub-base were created by element joints. The potential modelling of interface elements between the EPS blocks is only possible between the elements.

The created FE mesh presented in Figure 6.7, which met all the mentioned criteria, had 690 elements. Such an amount of elements was necessary to solve the discretisation problem in the non-linear elastic base material model. Unfortunately the calculation process was unstable because of high ratios of the stiffness of adjacent pavement materials. In order to solve this problem meshes with more elements were used as well, but the results obtained then were not essentially different from the results for the mesh with 690 elements, only the calculation time was much longer.

It is obvious that the FE element mesh had to be relatively fine in the areas with large stress gradients. Such large stress gradients occur in the upper part of the model, especially in the corner under the loading. In the remaining part of the mesh the elements may be greater. It would implicate the implementation of narrow and long
elements if the mesh refinements could not be applied. In such a case the limiting maximum ratio between the horizontal and vertical element dimension would result in a larger total number of elements in the model. Therefore another procedure was followed where the mesh was refined by establishing relationships between the displacements of the nodes on the joint side of the elements [12].

Figure 6.7
FE mesh of the DIANA axi-symmetric pavement model with implemented boundary conditions

6.4.2.2 Element types

The type of DIANA element used for the FE pavement analysis was the CQ16A element. This element is an axial-symmetric type applicable for modelling both linear-elastic and non-linear elastic material models [12].

CQ16A is a curved (C), quadrilateral (Q) axi-symmetric (A) element with a rectangular cross section. It has 8 nodes, shown in Figure 6.8, and in each node it has two degrees of freedom (in total 16). This element type has nine iteration points and in each of them it is possible to calculate the stresses and strains in the vertical (\(\sigma_y\)) and \(\epsilon_y\), radial (\(\sigma_r\) and \(\epsilon_r\)), and tangential direction (\(\sigma_{\theta}\) and \(\epsilon_{\theta}\)). The displacements in the horizontal (\(u_z\)) and the vertical (\(u_z\)) direction are available in the element nodes.

Figure 6.8
Quadrilateral axial symmetric element CQ16A
6.4.3 Material models

For the non-linear calculations on the pavement structures with an EPS sub-base both linear-elastic and non-linear elastic layer material models were simultaneously applied in the pavement models. In first instance all the layer materials had to be modelled as linear-elastic ones because the non-linear analysis started with a linear-elastic calculation. This was required in order to assemble the linear-elastic stiffness matrix in the beginning of the non-linear-elastic analysis. Non-linear material models were applied in the further analysis steps for the unbound granular roadbase and the sand layer which are placed on top of the EPS. The materials in the remaining pavement layers were assumed to be linear-elastic as it is shown in Figure 6.9.

6.4.3.1 Linear-elastic material model

Dense asphaltic concrete (d.a.c.), open asphaltic concrete (o.a.c.), gravel asphaltic concrete (g.a.c.), concrete, cement-stabilized sand, expanded polystyrene foam (EPS) and peat were represented by linear-elastic material models in the scope of this non-linear pavement analysis. Each of these materials had a modulus of elasticity $E$ [MPa], Poisson’s ratio $\nu$ [-] and density $\rho$ [kg/m$^3$]. The $E$ and $\nu$ values were also prescribed for the sand and the granular base material during the above mentioned initial linear-elastic calculation.

6.4.3.2 Non-linear material model

The granular materials have a stress-dependent stiffness. The stiffness increases with increasing stress. Young’s modulus and Poisson’s ratio are not appropriate for this type of materials. They establish a single stress-strain relation which is in contrast with the results of cyclic loading triaxial tests. A new approach is the material model with stress-dependent stiffness parameters. Three parameters: $K_1$ [MPa], $G_1$ [MPa] and $n$ [-] describe the non-linear elasticity of a granular material in the DIANA mechanical model. The relation between the volumetric strains ($\varepsilon_v$) and the mean principal stress ($p$) and the relation between the shear strains ($\varepsilon_s$) and stresses ($q$) are defined by these parameters. $K_1$, $G_1$ and $n$ are obtained from the measured axial ($\varepsilon_a$) and radial ($\varepsilon_r$) strains during a triaxial test. A large number of different stress-combinations has to be applied in this
test where the deformations of the specimens should not exceed the elastic regime. The applied axial and radial stresses and the measured strains are input data for a regression analysis which results in the parameters $K_1$, $G_1$ and $n$ for that particular specimen of unbound material.

The stress and strain invariants are calculated following [14]:

$$p = \frac{1}{3} |\sigma_1 + \sigma_2 + \sigma_3|$$  \hspace{1cm}  \text{[eq. 6.1]}  

$$q = \frac{1}{\sqrt{2}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$$  \hspace{1cm}  \text{[eq. 6.2]}  

$$\epsilon_v = |\epsilon_1 + \epsilon_2 + \epsilon_3|$$  \hspace{1cm}  \text{[eq. 6.3]}  

$$\epsilon_s = \frac{\sqrt{2}}{3} \sqrt{(\epsilon_1 - \epsilon_2)^2 + (\epsilon_2 - \epsilon_3)^2 + (\epsilon_3 - \epsilon_1)^2}$$  \hspace{1cm}  \text{[eq. 6.4]}  

where:

$\sigma_1$, $\sigma_2$, $\sigma_3$ - principal stresses
$
\epsilon_1$, $\epsilon_2$, $\epsilon_3$ - principal strains

According to Timoshenko, for a linear-elastic material the relationships between these stresses and strains are [15]:

$$\epsilon_v = \frac{1}{K} p$$  \hspace{1cm}  \text{[eq. 6.5]}  

$$\epsilon_s = \frac{1}{3G} q$$  \hspace{1cm}  \text{[eq. 6.6]}  

where:

$K$ - bulk modulus [MPa]
$G$ - shear modulus [MPa]

Boyce [16] defined the relationships [eq. 6.5] and [eq. 6.6] for a material with a stress-dependent stiffness as follows:

$$\epsilon_v = \frac{1}{K_1} p^n (1 - \beta \frac{q^2}{p^2})$$  \hspace{1cm}  \text{[eq. 6.7]}  

$$\epsilon_s = \frac{1}{3G_1} p^{n-1} q$$  \hspace{1cm}  \text{[eq. 6.8]}  

with:

$$\beta = \frac{K_1 (1 - n)}{6G_1}$$  \hspace{1cm}  \text{[eq. 6.9]}
Test Pavements Analysis

The non-linear material parameters $n$, $K_1$ and $G_1$ have the next meaning:

- $n$ - defines to what extent the stiffness modulus depends on the mean stress $p$. A linear-elastic material has a value $n = 1$. For a granular material generally this parameter has values in the interval $0 < n < 1$
- $K_1$ - reference bulk modulus [MPa]
- $G_1$ - reference shear modulus [MPa]

The procedure for the determination of the above mentioned non-linear parameters of an unbound gravel material consists of triaxial testing of the considered material and use of the measured strains in a regression analysis which results in non-linear parameters.

The three described parameters for a non-linear elastic material are not suitable in the case of occurring tension stresses in the roadbase. In that case the behaviour of the unbound granular materials is defined by an $E$ modulus and a Poisson’s coefficient. Therefore the complete input data set for the non-linear elastic material model is as follows:

- $G_1$ - reference shear modulus [MPa]
- $K_1$ - reference bulk modulus [MPa]
- $n$ - constant for the degree of non-linear elasticity [-]
- $E$ - modulus of elasticity in case of tensile stresses; taken as 1 MPa
- $\nu$ - Poisson’s ratio in case of tensile stresses [-]

6.4.4 Modelling of the load

The pavement structure was loaded by its dead weight and by a 50 kN load on a circular area with a 300 mm diameter. As first step only the stresses, strains and displacements due to the dead weight were calculated. Next the 50 kN load was added and the total stresses, strains and displacements were determined. Finally the results of the total loading minus the values for the dead weight only yield the stresses, strains and displacements caused by the 50 kN load which simulated the traffic loading.

The traffic load was applied in steps to make the convergency during the non-linear calculation procedure easier. The first step was a $0.4 \times 50$ kN load, the second step was another $0.4 \times 50$ kN load and the third step was the remaining $0.2 \times 50$ kN. In the case of the pavement models with interface elements the load was divided in even smaller parts and the calculations were performed in more than three steps.
6.4.5 Axial symmetric FE analysis of some pavement structures

6.4.5.1 Pavement classification and divergence problem

In order to be able to analyze the effects of an EPS sub-base on pavement performance as well as the effects of cement treated capping layers, two pairs of flexible pavement structures were analyzed. The first pair contained the pavement models with an EPS sub-base and a sand sub-base respectively. The second pair consisted of the pavement structures with and without a load-distributing concrete capping layer above the EPS sub-base.

The pavement pairs were characterized by identical layer thicknesses in the pavement models differed from each other with respect to the material in a pavement layer. The pavement structure characteristics, analysis procedure and calculation results for each of the pavement pairs are discussed in separate paragraphs. The comparison between the analyzed pavements was performed on the basis of the calculated vertical and horizontal stresses and strains in the axis of symmetry, i.e. in the vertical axis through the loading centre.

A large number of calculations preceded to those which results are presented. Pavement structure models with the implemented model of an EPS sub-base, with high deformation values combined with big differences in stiffness between the layers, appeared to be problematic cases for analysis by means of the DIANA program (release 3.2). Divergence during the calculation process made the completion of calculations impossible for several pavement structure models. Especially the interesting pavement structures with a thin asphalt top layer and an unbound roadbase without a load-spreading (cement-treated) capping layer on top of the EPS sub-base were difficult to analyze for this reason. Implementation of the EPS layer with a modulus of elasticity of 5 MPa led to high deformations in the considered pavement model under the standard loading of 50 kN and divergence. This problem was previously unknown. Later, the same divergence problems also occurred in pavement structures without the EPS sub-base.

In an attempt to achieve convergence of the calculation process, the loading was divided and brought step by step onto the pavement model. The calculation procedure converged in the beginning but, at a certain load, divergence occurred again. The number of elements was increased but, even then, no logical results could be obtained. Neither the use of more load steps instead of one-step loading nor the change of the iteration method was successful. Even the use of the mesh with a very large number of elements (1659) did not give adequate results: the calculation procedure remained unstable. Due to the enormous increase of required calculation time, further modelling was suspended.

The calculated stresses and strains in the axis of symmetry pointed out that the non-linear material model used for the unbound material in the roadbase caused the divergence problem. The loading value which caused this divergence was lower in the case of a thin asphalt layer, which implicated the occurrence of higher stresses in the
unbound base layer. It was a handicap in the analyses because these pavement structures (with thinner asphalt layers) offer interesting information about the maximum strain and stress values that could be expected in an EPS sub-base.

Solutions for the divergence problem were either the input of a higher E-value for EPS, the implementation of a thicker asphalt layer or implementation of a concrete layer on top of the EPS sub-base. However, all three ways impose limitations on the choice of pavement structures, which can be analyzed. The use of a somewhat higher EPS elasticity modulus enables, in contrast to the other two solutions, a free choice of layer thicknesses in the modelled pavement structures. However, one has to keep in mind that in this way stresses and strains in the pavement structures will be found, that are somewhat different from the values for the actual $E_{EPS}$ of 5 to 10 MPa.

The applied pavement model, for the structures with the unbound roadbase directly over an EPS sub-base, had an E-value of 15 MPa or higher for EPS. This is the minimal modulus of elasticity which allowed a stable calculation with DIANA. In the models of the pavement structures with a rigid layer between the unbound roadbase and the EPS layer, the value of 5 MPa could be used for the EPS modulus of elasticity.

6.4.5.2 EPS sub-base vs. sand sub-base

The analyzed pavement structures with an EPS sub-base (marked as N°1 in the further text) and a sand sub-base (marked as N°2) shown in Figure 6.10 had a 200 mm thick asphalt package (40 mm dense asphaltic concrete, 40 mm open asphaltic concrete and 120 mm gravel asphaltic concrete). The roadbase consisted of a 300 mm thick lava layer and a 100 mm sand layer. The sub-base of EPS or sand was 2.0 m thick.

The elasticity moduli of the pavement layers had the following values: $E_{d.a.c.} = 2,000$ MPa, $E_{o.a.c.} = 2,300$ MPa, $E_{o.a.c.} = 7,500$ MPa, $E_{EPS} = 15$ MPa and $E_{peat} = 30$ MPa. The material models used for unbound base materials were non-linear elastic with the parameter values $K_1 = 470.23$ MPa, $G_1 = 293.42$ MPa and $n = 0.39$ for lava, and $K_1 = 209.45$ MPa, $G_1 = 279.19$ MPa and $n = 0.36$ for sand [14]. The E values for the asphaltic concretes were determined by means of the nomograph for the determination of the stiffness modulus of bituminous mixes [17]. The stiffness modulus of the bitumen as well as the volume percentages of the mineral aggregate and the bitumen were used as input data.

Fig. 6.10 - Layer materials and thicknesses in the pavement structures N°1 and N°2
asphalt temperature for design purposes of 20°C in the Netherlands [18] was used as the reference temperature.

The vertical and horizontal stresses in the axis of symmetry of the analyzed pavement structures are shown separately in the Figures 6.11 and 6.12. In Figure 6.11 the vertical stresses due to the dead weight are shown and also the stress values due to the total loading, the dead weight plus the 50 kN force. In the upper layers the dead weight is relatively small and the stresses due to the 50 kN force (simulated traffic loading) dominate. In the deeper laid layers an opposite situation can be observed. There, the vertical stresses are much more the result of the dead weight than of the surface loading.

**Figure 6.11** - Vertical stresses in the axis of symmetry in the pavement structures N°1 and N°2 respectively

The extent to which the implementation of an EPS sub-base reduces the dead weight in the pavement can be observed in Figure 6.11. The total vertical stress below the EPS sub-base is 15.2 kPa (at a depth of 2.6 m in the structure N°1) while the accompanying vertical stress below a sand sub-base is 48.6 kPa (N°2). This means a stress reduction of 70% for the subsoil.

The calculated total vertical stresses at the top of the roadbase (at the depth of 0.2 m) are also different for the pavement N°1 (σ_v = 48.6 kPa) and the pavement N°2 (σ_v = 66.9 kPa). At the bottom of the lava layer, these values are σ_v = 28.6 kPa (N°1) and σ_v = 43.2 kPa (N°2). Lower stress values in the lava, an unbound material with a stress-dependent behaviour, implies its 'lower' effective stiffness in pavement structure N°1 above the EPS sub-base.
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Fig. 6.12 - Horizontal stresses in the axis of symmetry in the pavement structures N°1 and N°2 respectively

The stress lines in Figure 6.12 present the horizontal stresses ($\sigma_x$) which are a result of the simulated traffic loading. Most interesting is the value at the bottom of the asphalt layer (at 0.2 m depth). $\sigma_x = 2,040$ kPa and $\sigma_x = 1,758$ kPa were found in the pavements N°1 and N°2 respectively. A larger value of the horizontal asphalt stress above the EPS sub-base is a result of the reduced 'effective' stiffness of the roadbase. Accordingly, the asphalt layer bends more under the loading because of a reduced support from underneath. It results in higher horizontal stresses at the bottom of the asphalt layer.

In Figures 6.13 and 6.14 the vertical and horizontal strains in the considered pavement structures are shown. The maximum calculated vertical strain in EPS is equal to 0.18%. According to the calculated (small) strain value it seems likely that exclusively elastic deformations will occur in the EPS sub-base, at least in similar pavement types. Larger vertical strain values in the subsoil in the pavement N°2 are a result of the heavier upper layers in this traditional pavement structure.

Figure 6.13 - Compressive vertical strains in the axis of symmetry in the pavement structures N°1 and N°2 respectively
An important design criterion for the determination of the design life of an asphalt pavement structure is the horizontal strain at the bottom of the asphalt layer. These values are $\epsilon_r = 178 \, \mu \text{m/m}$ and $\epsilon_r = 154 \, \mu \text{m/m}$ in the pavement N°1 and N°2 respectively. Such a strain difference of 15% results in a much larger difference in the allowable number of load repetitions (N) because of a logarithmic relation between $\epsilon_r$ and N. Using the fatigue relation which is developed for the standard Dutch gravel asphaltic concrete [18], pavement structure N°2 (N $\approx 0.5 \times 10^8$) would have twice the design life of pavement structure N°1 (N $\approx 0.25 \times 10^8$).

In order to investigate the influence of the EPS thickness on the stresses and strains in the pavement layers, the calculations were also performed on three sub-variants of the model N°1. These three sub-variants were only different with respect to the EPS thickness. The implemented EPS layers were 0.5 m, 1.0 m and 2.0 m (case N°3) thick. However, the calculated stress and strain values in the upper pavement layers were almost the same in all three cases. This indicates that the thickness of the EPS sub-base has only a marginal influence on the pavement behaviour under loading and on the pavement design life. This conclusion is limited to the EPS types with low densities and thus low stiffnesses.

6.4.5.3 Pavement structures with and without a concrete capping layer

The analyzed pavement structures N°3 and N°4 have a 220 mm asphalt tolayer and a 1.0 m EPS sub-base as shown in Figure 6.15. The main difference between these two pavement models concerns the roadbase. The structure N°3 has a pure gravel roadbase of 340 mm on top of the EPS sub-base. The roadbase of the pavement structure N°4 consists of a gravel base of 200 mm and a 140 mm cement-treated capping layer on top of the EPS sub-base. The layer thicknesses were adopted from the test sections at BASf (see subsection 6.2.2) The used values for the elasticity moduli were back-calculated from
the falling weight measurements carried out on these structures (see Table 6.3). The following $E$ values were used in the pavement $N^3$: $E_{\text{asph}}$ is 9,000 MPa, $E_{\text{ctl}}$ = 6,700 MPa, $E_{\text{EPS}}$ = 34 MPa and $E_{\text{subsoil}}$ = 41 MPa. The layers in pavement $N^4$ had $E$ values as follows: $E_{\text{asph}}$ = 9,000 MPa, $E_{\text{EPS}}$ = 26 MPa and $E_{\text{subsoil}}$ = 36 MPa. The unbound roadbase materials were represented in the DIANA-calculations by the non-linear elastic lava model (see paragraph 6.4.5.2).

The vertical and horizontal stresses in the axis of symmetry of the analyzed pavement structures $N^3$ and $N^4$ are shown separately in Figures 6.16 and 6.17 respectively. Considering the difference between the pavements $N^3$ and $N^4$, the stresses in the unbound roadbase layer above the cement treated layer are higher than in the unbound material laid directly above EPS. Despite the relatively high $E$-values assumed for EPS, this layer does not provide such a good support to the unbound lava as the capping layer does in $N^4$. Consequently, in pavement $N^3$ lava has a lower 'effective' stiffness and this layer gives thus a reduced support to the upper asphalt layer.

![Fig. 6.15 - Layer materials and thicknesses in the pavement structures $N^3$ and $N^4$](image)

![Fig. 6.16 - Vertical stresses in the axis of symmetry in the pavement structures $N^3$ and $N^4$ respectively](image)
The horizontal stress graphs in Figure 6.17 show that the cement-treated capping layer in pavement structure N⁴4 is also subjected to bending to a certain extent. It illustrates the load spreading occurring in the cement-treated layer. The support this layer offers to the overlaying upper roadbase layer enables the development of the all-round confining stresses in the unbound material. It results directly in a higher 'effective' stiffness of lava and indirectly in an increased support to the asphalt layer. The final effect is a decrease of the horizontal stresses at the bottom of the asphalt layer for approximately 15% ($\sigma_r = 1379$ kPa in N⁴3 instead of $\sigma_r = 1,10$ kPa in N⁴4).

In Figures 6.18 and 6.19 the vertical and horizontal strains in the axis of symmetry of the considered pavement structures are illustrated.
The values of the total vertical strain at the top of the roadbase (lava) indicate the extent to which the 'effective' stiffness of the unbound material is improved by implementation of the cement-treated capping layer. The vertical strain in the pavement structure N°4 is only 17% higher than in N°3 (495 and 423 µm/m respectively) despite a difference of 50% in the vertical stresses (27.2 and 40.6 kPa respectively).

**Fig. 6.19 - Horizontal strains in the axis of symmetry in the pavement structures N°3 and N°4 respectively**

The horizontal strains at the bottom of the asphalt layer are $\varepsilon_r = 105$ and $\varepsilon_r = 90$ µm/m in pavements N°3 and N°4 respectively. The same arguments as in the previous paragraph about the strain difference are also valid here. The strain difference is about 15% but expressed in the allowable number of standard axle load repetitions (N) it becomes much more. Following the same asphalt fatigue curve used in the previous subsection, the pavement structure N°4 with a cement-treated capping layer can sustain twice as many standard axle load repetitions ($N = 3.6 \times 10^9$) than N°3 with a pure unbound roadbase ($N = 1.8 \times 10^9$). In other words, the implementation of a load-spreading (cement-treated) capping layer improves the performance of a pavement structure with an EPS sub-base substantially.

### 6.4.6 Concluding remarks on axial symmetric pavement analysis

Based on the results of the analyses that were discussed in the previous subsections, the following conclusions are made:

The total vertical stresses at the top of the unbound roadbase above the EPS sub-base are lower than above a sand sub-base. Lower stress values in the unbound material with a stress-dependent behaviour yield a lower 'effective' stiffness if this material is laid directly above the EPS sub-base. Accordingly, the asphalt layer bends more under the
loading because of a reduced support from underneath which results in higher horizontal stresses and strains at the bottom of the asphalt layer.

The horizontal strain at the bottom of the 200 mm thick asphalt layer, an important design criterion for the determination of the design life of an asphalt pavement structure, is approximately 15% higher in case an EPS sub-base is used instead of a sand sub-base below an unbound roadbase. Such a strain difference results in a reduction of the allowable number of standard axle load repetitions by approximately 50% if the EPS sub-base is applied.

The thickness of the EPS sub-base has only a marginal influence on the pavement behaviour under loading and on the pavement design life.

The application of a cement-treated capping layer above the EPS sub-base offers the support to the overlying upper roadbase layer and enables the development of the all-around confining stresses in the unbound material. This immediately results in a higher 'effective' stiffness of the unbound material and in an increased support to the asphalt layer. The final effect is a decrease of the horizontal stresses and strains at the bottom of the asphalt layer and a significantly longer design life of the pavement structure.

From the maximum calculated vertical strain in EPS (equal to 0.18%) it can be concluded that exclusively elastic deformations will occur in the EPS sub-base, at least in case of a 200 mm thick asphaltic concrete toplayer.

6.5 FE ANALYSIS OF THE BAS$t TEST SECTIONS

6.5.1 Test pavement modelling

The BAS$t test sections were simplified to an axial symmetric body. The created pavement models were loaded by both the dead weight of the pavement layers and by a 50 kN standard wheel loading which simulated traffic. This standard wheel loading was distributed over a circular contact area with a diameter of 300 mm. This diameter is identical to that of the FWD plate which enabled the comparison of measured FWD deflection bowls to calculated ones.

The non-linear material model simulated the stress dependent behaviour of the unbound granular materials of the roadbase (gravel) and the sand layer below the EPS sub-base. For the other pavement layers (asphalt, concrete, cement stabilized sand, EPS and loam) a linear-elastic model was used where the material was characterized by its Young's modulus $E$ and Poisson's ratio $\nu$.

The original gravel material from the BAS$t test pavement structures was tested in a triaxial set up, shown in Figure 6.20, at the Laboratory for Road and Railroad Research of the Delft University of Technology. The water content of the tested material was equal
to the optimal value (10%) which was derived from Proctor test results. The construction and compaction of the specimen was done according to the guidelines given by Sweere and Vogelzang [19]. The degree of compaction was 102% equal to the average value found for the gravel layers in the BASt test pavement structures.

From the test results, the following values of the Boyce model were calculated: $K_v = 213.95$ MPa, $G_v = 528.24$ MPa and $n = 0.32$. For the sand in the test pavement structures the values of the Dutch 'Oosterschelde' sand ($K_v = 279.19$, $G_v = 209.45$ and $n = 0.36$) were used.

Young's moduli for the linear-elastic materials (asphalt and cement-bound materials) were obtained experimentally (see subsection 6.2.2). The values of Young's modulus for the subgrade (loam) and EPS were calculated in the course of the DIANA model validation.

6.5.2 Model validation

The DIANA finite element model for the BASt test pavements was validated using data from FWD measurements and measured horizontal strains at the bottom of the asphalt layers. These measured values were compared to the respective calculation results.

To simulate the influence of dynamic loading, Young's moduli values were used for the asphalt layers of the DIANA model, which corresponded to the real load frequencies and the real temperatures as occurred during the measurements. The asphalt Young's moduli were obtained experimentally for different temperatures and frequencies by means of a four-point bending test.

The FWD loading time of 0.02 s with a half-sine wave form correlates with a 25 Hz sinusoidal loading in the four-point bending test. At the moment of the FWD measurements, the temperature in the laboratory reached 17.5°C. For these temperatures and this frequency, the experimentally-determined Young's moduli were equal to 9,500 MPa for the 40 mm asphalt wearing course and 11,300 MPa for the asphalt base course. These values were input data for all the DIANA calculations.

The intention was to use an E-value of 5 MPa for EPS but it appeared from the analysis that only for section 'GB' 3 an agreement between measured and calculated deflections
could be obtained by using this value. For sections 'CB'1 and 'SB'5, with the thick asphalt layer (220 mm) the calculated deflections were only in agreement with the measured ones in the case of Young's moduli for the EPS layer of 9 and 10 MPa respectively. In the same manner, E-values of 16 and 17 MPa were found for sections 'CB'2 and 'SB'6 respectively. The moduli of elasticity of the subgrade, which were iteratively determined, took values of 80 MPa for sections 'CB'1, 'CB'2, 'GB'3 and 'SB'5 and 100 MPa for section 'SB'6. For section 'GB'4 no solution could be generated due to divergence problems.

The measured and calculated deflection bowls are shown in Figure 6.21. A very good agreement was found for section 'GB'3. The calculated Young's modulus for EPS had the value which was also experimentally obtained. In the case of the more complicated pavement structures 'CB'1, 'CB'2, 'SB'5 and 'SB'6, the similarity between the measured and calculated deflection bowl existed only for the first part (0.5 m) with a dissimilarity up to 5% which increased many-fold at the distance of 2.0 m from the loading centre. The horizontal strains due to a passage of the 100 kN standard axle load were calculated using appropriate E-values for the asphalt layer for the temperature of 20°C.

![Figure 6.21 - Calculated and measured deflection bowls](image)

The calculated asphalt strains are shown in Table 6.4. From a comparison between the calculation results in Table 6.4 and the measurement results shown in Table 6.2 [20] one can conclude that an agreement within 20% was observed. Due to divergence of the calculation process, no results for the 'GB' test sections 3 and 4 could be obtained.

In Table 6.4 the horizontal asphalt strains as calculated for the FWD measurement are also given.
Test Pavements Analysis

<table>
<thead>
<tr>
<th>Test section</th>
<th>'CB'1</th>
<th>'CB'2</th>
<th>'GB'3</th>
<th>'GB'4</th>
<th>'SB'5</th>
<th>'SB'6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{sph}$ ← freq(v= 3km/h), $T=20^\circ$C</td>
<td>149</td>
<td>215</td>
<td>*</td>
<td>*</td>
<td>137</td>
<td>200</td>
</tr>
<tr>
<td>$E_{sph}$ ← freq(v=10km/h), $T=20^\circ$C</td>
<td>119</td>
<td>179</td>
<td>*</td>
<td>*</td>
<td>111</td>
<td>168</td>
</tr>
<tr>
<td>$E_{sph}$ ← freq(FWD), $T=17.5^\circ$C</td>
<td>78</td>
<td>124</td>
<td>101</td>
<td>*</td>
<td>74</td>
<td>118</td>
</tr>
</tbody>
</table>

* - no result because of divergence

Table 6.4 - Calculated horizontal strains at the bottom of the asphalt layer

6.5.3 Test sections axial symmetric analysis results

The use of non-linear material models gave the opportunity to analyze the influence of a load-distributing intermediate layer on top of the EPS sub-base on the unbound roadbase material. The calculated Mr-values were not constant over the entire thickness of the roadbase. Especially in the 'GB' pavement structures, there was a high gradient of the Mr-value in the unbound roadbase.

![Figure 6.22 - Resilient modulus, Mr, in the axis of symmetry in the unbound roadbase material of the BASl test sections](image-url)
The calculated Mr-values of unbound roadbase materials in pavement structures with a loadspreading cement-bound layer above the EPS sub-base are higher than those for unbound roadbases when such a cement bound layer is not present (see Fig. 6.22). The consequence is that in linear-elastic multi-layer calculations the same unbound roadbase material should be given a higher E-value in a pavement structure with a load-distributing layer than in a pavement structure without such a layer. Because there was a divergence of the calculation process, no results for the 'GB' section 4 could be obtained.

The calculated vertical strains in the EPS sub-base of all test pavements are shown in Figure 6.23. The highest strains were found in pavement structure 'GB'3. The maximum strain was equal to about 0.35%, where 0.1% was a consequence of the dead weight of the layers above the sub-base. The experimentally measured strain of EPS20 in the linear-elastic region was equal to at least 0.4% (see Chapter 5). If this strain value is taken as a design criterion for pavement structures with an EPS sub-base then it was met in all structures.

Horizontal strains in the EPS sub-base were practically negligible.

The maximal calculated vertical stress was about 27 kPa, of which 12 kPa was the consequence of the dead weight. This means that the stress difference of 15 kPa due to the wheel load is within the linear-elastic region (see section 5.6). The highest horizontal tensile stress had a value of 3.5 kPa. The calculated stress values confirmed that EPS in the BASt test pavement's sub-base could be considered as a linear-elastic material.

Figure 6.23 - Compressive vertical strain in the axis of symmetry in the EPS sub-base of the BASt test sections
6.6 CONCLUSIONS

- In the BASt experiment, the most suitable equipment for compaction of unbound material in pavement structures with an EPS sub-base was the plate vibrator. The static roller is less efficient than a vibrator, while a vibrating roller is unsuitable.

- Truck serviceability in the construction phase is unconditionally possible only on section 'CB'2 with a reinforced concrete capping layer above EPS and a 280 mm unbound roadbase on top of it. Loadings lower than 50 kN per wheel are allowed on roadbases of the sections 'CB'1 (with a concrete capping and a 200 mm thick unbound base layer), 'SB'5 and 'SB'6 (with a cement-stabilized sand layer above the EPS sub-base) while the sections 'GB'3 and 'GB'4 (with a roadbase of only unbound gravel material) should under no circumstances be subjected to heavy vehicles in the construction phase. It would result into high plastic deformations in the base, and therefore only vehicles with wheel configurations that results in low stresses might be allowed.

- The horizontal strain at the bottom of the (200 mm thick) asphalt layer, an important criterion for the determination of the design life of an asphalt pavement structure, is approximately 15% higher when an EPS sub-base is built-in below an unbound roadbase instead of sand. Such a strain difference results in an approximately two times lower allowable number of standard axle load repetitions, i.e. a two times shorter pavement design life if an EPS sub-base is applied.

- Because of its very low modulus of elasticity, the thickness of the EPS sub-base has only a marginal influence on the pavement behaviour under loading and on the pavement design life. The stress and strain values in the upper pavement layers are almost the same in pavement structures with different thicknesses of the EPS layer.

- Unbound roadbase materials have a lower 'effective' stiffness in pavement structures without a cement-bound load-spreading layer above the EPS sub-base compared to a structure with such a layer. Therefore, in linear-elastic calculations the assumed E-value for the unbound material should be lower when this material is laid directly over an EPS sub-base than in the case of an intermediate cement bound layer.

- Implementation of a cement-treated capping layer on top of the EPS sub-base substantially increases the design life of the pavement structure with an EPS sub-base. The application of such a layer is therefore recommended for pavement structures subjected to heavy traffic loading.

- 'GB' pavement structures, as applied at BASt, can sustain only a low number of standard load repetitions before cracking of the asphalt occurs.
Maximum vertical and horizontal stress and strain values calculated in the EPS layer of all the analyzed pavements under a standard 50 kN load do not exceed the experimentally determined boundaries of the elastic region of the EPS. It confirms the conclusion that EPS in the BASt test pavements' sub-base can be considered as a linear-elastic material.

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CHAPTER 7

MEASUREMENTS ON BLOCK PAVEMENT STRUCTURES

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7.1 INTRODUCTION

There is a lack of knowledge of the structural behaviour of pavement structures with an EPS sub-base compared to common structures with traditional sub-base materials. That is valid for concrete block pavement structures in particular. Test sections for research into the behaviour of pavements with an EPS sub-base, described in the previous chapter and literature [1], were flexible pavement structures with a toplayer of asphaltic concrete. The biggest EPS project in The Netherlands, at Capelle a/d IJsel near Rotterdam, was the reconstruction of concrete block pavement structures by laying 35,000 m³ EPS in the sub-base [2]. A similar reconstruction, using a smaller quantity of (about 9,000 m³) EPS, was carried out in the autumn of 1990 in the parish of Overwetering at Stolwijk near Gouda. This last project was used to perform a series of Falling Weight Deflection (FWD) measurements in order to follow systematically the performance of these structures from the date of reconstruction in December 1990 until February 1993. The measuring program consisted of deflection measurements (approximately every three months) at 39 points, divided over three roads. Additionally, penetration tests and visual inspections were carried out once. The results of these analyses are described in this chapter.

In Section 7.2 the measuring program is discussed. Firstly, the reasons for the choice of FWD measurements and its basic principles are listed, including an explanation about the use of the measuring results. Secondly, the characteristics of the FWD measurements and data are reviewed. The next section contains some information about the penetration test and its purpose. Finally, the visual inspection to observe the rut depth and other visible defects at the pavement surface is described.

Section 7.3 contains a description of the location and details about the three sections where FWD measurements have been done. The traffic situation at the test sections is qualitatively described and some rough figures about the traffic loading are presented. Furthermore soil investigation results, including those of dynamic cone penetration tests, the cone penetration tests and the borings, are presented. The design procedure chosen by the road authority is described and layer materials and thicknesses of the tested concrete block pavement structures are shown. This section is completed by some remarks on the relevant findings of the performed visual inspection.

In Section 7.4 the FWD measurement results are presented. Some measuring sites were chosen as representative for the sections considered. For these selected sites, graphical representations are made of the FWD deflections as a function of time. It enables one to obtain an insight into the structural pavement behaviour in time and the influence of the traffic. The FWD results were used for a back-calculation analysis which resulted in the moduli of elasticity (E-moduli) of the pavement layers. The linear-elastic multi layer models used for this back-calculation of the layer E-moduli are described in the next section. Finally, the back-calculated layer E-moduli are listed and discussed.
In Section 7.5 general conclusions are drawn, which refer to the general condition of the block pavement structures, differences between the characteristics of the sections and remarks on the E-values found for the unbound materials and for EPS.

### 7.2 MEASURING PROGRAM

In order to gain an insight into the structural behaviour of concrete block pavement structures with an EPS sub-base, FWD measurements were periodically done on 39 locations in total on three residential streets with an EPS sub-base, at Stolwijk. The measurements have been performed about every three months from October 1990 (completion of the pavement structures, i.e. opening to traffic) to February 1993.

To get more information about the bearing capacity of the local soil layers, Dynamic Cone Penetration (DCP) Tests were carried out in the polder as well as at three locations inside the parish of Overwetering. The results enabled, through calculation of the California Bearing Ratio (CBR) [3], to get an indication of the bearing capacity of the soil in the area under investigation. Furthermore, the results of the soil investigation program [4], performed by a geotechnical consultant in the parish of Overwetering in August 1988, were studied. The program consisted of Cone Penetration Testing (CPT) at 15 points, making boreholes (4 points) and laboratory testing of the obtained undisturbed soil samples.

#### 7.2.1 FWD measurements

Measuring the surface vertical displacements (deflections) due to a dynamic loading provides input data for the back-calculation of the modulus of elasticity of the individual pavement layers. The back-calculated layer moduli give an impression about the actual bearing capacity of the pavement structure.

In the 'back-calculation' of the layer E-moduli the pavement structures were modelled as a linear-elastic layered system. Although in reality unbound materials show a stress dependent stiffness behaviour, they were modelled linear-elastically in this analysis. This is done to comply with the analysis methods which are used in practice. For this reason it is necessary to measure the deflections caused by a load which is similar in magnitude and duration to that of a real moving truck wheel. However, FWD measurements on the block pavement structures discussed in this chapter, were carried out with a 25 kN force instead of the usual 50 kN. The reason for this is that the measured deflections due to a 50 kN load were out of the range (approximately 2 mm) of the geophones. The induced deflections are registered by six geophones placed from the middle of the loading plate up to a distance of 1.8 m from the plate centre. The complete back-calculation results, including the choice of representative points on the test sections, are described in subsection 7.4.3.
7.2.2 Soil Investigation

As mentioned before part of the soil investigation that was carried out consisted of Dynamic Cone Penetration Tests (DCP Tests) and Cone Penetration Tests (CPT). The DCP Tests procedure consists of driving a cone attached to a set of rods into the soil by means of a falling weight. In this way, valuable information about soil conditions can be collected relatively simply and quickly. The test is suitable for fine soils ranging from silt to fine sandy soils for which a correlation has been established between the penetration of the cone per blow and the CBR [3]. As mentioned earlier, the CBR value gives an indication of the bearing capacity of the considered soil. The relationship used for conversion of the penetration (mm/blow) into CBR-values is presented in section 7.3.

Cone Penetration Tests (CPT) penetrating to NAP-20 m were done and boreholes to NAP-12 m were made by a specialized company. The abbreviation NAP stands for Normal Amsterdam Level meaning the average sea level. At the time of investigation the site level varied between NAP-1.93 m and -1.36 m. The locations of CPT's (1, ..., 9) on the three streets where the deflections were measured, are marked in Figure 7.2. The boreholes were made at the points 3, 5 and 9.

The CPT produces a cone resistance $q_c$ [MN/m²] as a measure for the strength of the various soil strata. Furthermore, the local friction $f_s$ [MN/m²] is registered along the adhesion jacket placed close above the cone. To determine $q_c$, the force exerted to push only the cone was measured. $f_s$ is calculated from the force needed to overcome the friction along the jacket, which is equal to the difference in force between pushing down only the cone and the cone together with the adhesion jacket. The cone resistance was measured by means of the so-called electrical method. The cone and jacket (friction) surface and the cone shape are standardized [5]. An example of Cone Penetration Test performed in the parish of Overwetering in Stolwijk was already shown in Figure 1.3.

The ratio between $q_c$ and $f_s$ is characteristic for each type of soil, and is called the friction ratio. Accordingly, the CPT can replace, in many cases, the boring to obtain a soil-profile. In Figure 7.1 the relationship between $q_c$, $f_s$ and the friction ratio is illustrated as it has been found [6] for conditions in

![Fig. 7.1 - Relation between the friction ratio and the type of soil for the electrical adhesion jacket cone](image-url)
The Netherlands. Anyhow, boreholes were made on the considered locations, with undisturbed soil samples taken for additional laboratory testing. Laboratory investigation concerned settlement parameters, density and water content in the soil, particularly in the peat.

7.2.3 Visual Inspection

In order to survey the actual conditions of the concrete block pavement structures in Stolwijk, a visual inspection has been performed in May 1993 after all FWD measurements on all three test sections were done. The inspection consisted of measuring rut depths, joint widths and local unevenness, and the observation of the quality of concrete blocks and the position of the kerbs, following the procedure which is prescribed in the literature [7].

Rut depth is the value of permanent vertical deformation in the cross section measured under a 1.2 m long straight edge laid down across the street. By means of this straight edge also the extent of the local unevenness was determined. Furthermore, the joint width between concrete blocks was measured. As for the quality of the concrete blocks, attention was paid to signs of breaking, edge spalling and weathering of the blocks. Finally, the conditions of the kerbs were inspected. All relevant observations pertaining to the visual inspection of the measuring site are presented in subsection 7.3.4.

7.3 DESCRIPTION OF THE MEASURING SITE

The area around Gouda is well known in civil engineering in The Netherlands because of its subsoil with a very low bearing capacity. The subsoil consists of about 10 m thick peat layers over a deep sand. A consequence is the occurrence of long-term settlements on the local roads. Even the roads which were built many years ago still have to be reconstructed periodically in order to bring their level back to the same level as the houses and the garage doors. In contrast with the roads, the buildings in this area are founded on piles and are not subjected to settlements. In other words, reconstruction of concrete block pavement structures by applying an extra sand layer below the blocks means an increase of the existing load on the weak subsoil which results in additional settlement of the very thick compressible peat layers. The roads continue to settle, become uneven again, also significant height differences between buildings and surrounding pavements are developed again, and within a relatively short period of time maintenance is needed. Because of the very low bearing capacity of the subsoil this problem returns after a few years.
7.3.1 Layout of the Measuring Site

The road authorities in the parish of Overwetering at Stolwijk (10 km south of Gouda) have been confronted with the problem described above. The reconstruction of the concrete block pavement structures was necessary because of the obtained settlements. Experience and soil-mechanical research did not promise anything else than the need for a new levelling after a few years, as many times before. The measured settlements amounted up to 0.2 m per year. Besides the costs of repeated road reconstruction, the on-going settlement would increasingly irritate the local inhabitants, which would be no recommendation for the responsible municipal engineers. Faced with this problem the engineers chose a non-familiar but in practice already proven solution: the use of the light-weight EPS in the sub-base. It was the only economic low-maintenance long-term solution. Therefore the pavement structure with an EPS sub-base was considered to be a cost effective design. The high initial costs will be compensated by savings on the maintenance costs.

The complete parish of Overwetering is shown in Figure 7.2. The three streets, where FWD measurements were carried out periodically, are indicated in the plan: Bilwijkweg, Dahliastraat and Korenbloemstraat. The locations ★ (I, ..., IV) indicate the locations of the DCP Tests while the points ▼ (1, ..., 9) signify where soil characteristics were known from literature [4].

Figure 7.2 - Plan of the parish of Overwetering at Stolwijk with measuring sections
7.3.2 Traffic Loading

The traffic intensity on the considered pavement structures is very low. Only a few trucks per day can be expected in this quiet residential area. The rest of the traffic consists of person cars. In order to get an insight into the traffic intensity, the vehicles passing the Bilwijkerweg were counted; the Bilwijkerweg was chosen because it is the most heavily loaded road in the parish. An unobstructed traffic flow in both directions is not possible on most parts of the Bilwijkerweg. Its cross section, 6 m wide with a narrow parking lane of 1.2 m, is reduced because of permanently parked cars along the southwestern lane. Consequently, the middle part of the road cross-section is used by all passing vehicles, at least on part of the Bilwijkerweg. The rut depth, the major design criterion for small element pavement structures, is thus a result of the total traffic loading in both directions. The daily number of passing vehicles was estimated to be 500 cars and light goods vehicles, 60 two-axle heavy goods vehicles (HGV’s), 5 three-axle HGV’s, 5 four-axle HGV’s and 30 one-axle agricultural semi-trailers for both directions together. In the case of the other two streets, the Korenbloemstraat and the Dahliastraat, the traffic was less intensive than on the Bilwijkerweg and consisted only of the person cars of the local people.

7.3.3 Soil Characteristics

In order to determine future settlements in the area of interest, a soil investigation was ordered by the municipal road authorities in August 1988. The site level varied between NAP-1.93 m and -1.36 m, while the average groundwater level was at NAP-2.02 m. A desirable street level is 0.6 m above the water table which corresponds to the level at NAP-1.42 m. The average annual settlement, ascertained by observation after a previous street level correction, amounted to 0.2 m per year at that time. Therefore, in spite of a recent reconstruction with a 0.5 m thick sand layer the street level was actually only between 0.1 and 0.6 m above the groundwater level.

The Dynamic Cone Penetration (DCP) Tests that were described in subsection 7.2.2, were performed both in the polder outside the parish of Overwetering (point I) and inside the parish (points II, III and IV). Test locations are indicated in Fig. 7.2. The first DCP Test was carried out in the polder in order to provide data about original soil characteristics before overlaying the soil by sand and preparing the site for building. The three remaining DCP Tests provided data in the surrounding area near the streets where FWD measurements were performed regularly.

The penetration X in mm per blow, obtained from the DCP Tests, was converted into the California Bearing Ratio (CBR) using the following relationship:

\[
\log CBR = -1.31 \log X + 2.68 \quad \text{[eq. 7.1]}
\]
According to literature [3] and [8], the next three limiting conditions are related to the relationship \[eq. 7.1\]:

a) it is limited to fine soils (ranging from silt to fine sandy soils < 0.2 mm - particle diameter)

b) depth of penetration up to 2 m

c) penetration per blow \((X)\) less than or equal to 40 mm

When these conditions are fulfilled the 95% confidence interval for the real CBR is between 0.7 and 1.4 times the calculated CBR values. DCP Test results are commonly entered in a chart where the CBR is on the horizontal axis and the corresponding depth on the vertical axis. Such a diagram with the DCP Test results for the original soil in the polder (I), and on the other three locations (II, III and IV) inside the parish of Overweerding, is shown in Figure 7.3. It was assumed that the local sand fulfills the condition regarding the maximum particle diameter.

![Diagram of CBR vs Depth](image)

**Fig. 7.3 - Dynamic Cone Penetration test results for original soil in the polder (I) and in the parish (II, III & IV)**

The DCP Test results pointed to a very low bearing capacity of the original subsoil in the polder. The relatively higher CBR values found within the parish, belong to the sand layers which were laid in the scope of preparing the site for building and/or previous road reconstructions. Variations in CBR within the sand layers are the effects of compaction. The sand layers have very variable thicknesses. Its thickness was 0.5 m at point IV and already 1.4 m at point III, although the horizontal distance between these points was less than 10 m.

The soil profile, determined from the cone penetration tests and boring results, showed a sand layer with a thickness from 0.6 to 3.7 m. Upper sand layers, created due to repeated overlaying, were thicker under the Bilwijkeweg with a thickness in the range of 1.5 to 3.7 m. Under the Korenbloemstraat and Dahliastraat the sand thickness varied between 0.6 and 1.4 m.

Below the sand a peat layer was found to the depth of NAP-10 m. Between NAP-10 m and NAP-13 m peat, sand and clay were found. The pleistocene sand stratum with a good bearing capacity was found at NAP-13 m at all investigated locations.

The laboratory investigation on the so-called undisturbed peat specimens indicated a very low dry-matter content which corresponds to a high potential compressibility of this layer. Average water content in the peat amounted to approx. 500% m/m (related
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to amount of dry material). This high water content indicates a high void content and so the conclusion can easily be drawn that additional surcharge due to the levelling layers will result in substantial settlement of the surface level in the next couple of years.

The settlement process in the peat, caused by additional loading, can be divided into two components: the above-mentioned short-term primary settlements (consolidation), and long-term secular (secondary) settlements ('creep'). By each new overlaying not only the street level will settle to the previous level in a short time, but also the total secular long-term settlements will increase.

Settlement tests on so-called undisturbed boring specimens predicted substantial secular settlements to be expected in Overwetering. The results predicted for the Bilwijkerweg additional Secondary Settlements (SS) between 140 and 300 mm during the next 25 years; a loading reduction of 6 kPa by means of replacing the upper heavy layers and use of a light foundation material, would lead to Restricted Settlements (RS) between 50 and 180 mm during the mentioned period. In the case of both Dahliastraat and Korenbloemstraat the expected SS varied between 100 and 150 mm while the RS were estimated between 0 and 50 mm.

All in all, overlaying by means of a (heavy) sand layer would cause not only the short-term primary settlements but it would also increase the long-term secondary settlements. Within a few years the situation would be worse than the original state before overlaying. Use of a light foundation material and reduction of the existing dead weight above the peat layer would limit the additional settlements.

7.3.4 Concrete Block Pavement Structures

Weight balance and budget limitations were the two major criteria for the specific design of the concrete block pavement structures at Stolwijk. The boundary condition was the required minimum level of 0.6 m above the groundwater table. Originally, the loading reduction of 6 kPa was taken as the design starting-point. This value was determined within the scope of the soil investigation, and was judged to be sufficient for a significant reduction of ongoing long-term settlements.

The weight-balance calculations consisted of a comparison of the stresses in the subsoil caused by the weight of the existing pavement layers and the stresses due to the weight of the new pavement structure. The depth of the bottom of the EPS sub-base was established as the reference level. The guidelines prescribed in the literature [9] were used for calculation of the concerned stresses. Though the registered groundwater level varied between NAP-2.32 m and NAP-1.84 m within the considered area, the (polder) level at NAP-2.02 m was assumed as an average for the whole parish. Ensuring a distance of 0.6 m between the road surface and the groundwater level resulted in a design level for the surface of the small element pavement (new street level) at NAP-1.42 m. The results of the stress calculations in the subsoil ($\sigma_u$) under the existing and designed pavement structures are shown in Figure 7.4.
Building-in an EPS sub-base of 0.8 m would enable a total stress reduction $\Delta \sigma_{ss}$ of 6.47 kPa. That is more than $\Delta \sigma_{ss} = 6$ kPa which should be enough according to the laboratory testing in the scope of soil investigation [4] for a significant reduction of the ongoing settlements. Material densities used for the weight balance calculations corresponding to the saturated materials state were: $\rho_{CB} = 2,400, \rho_{sand} = 2,000$ and $\rho_{BFS} = 2,200$ kg/m$^3$ [9]. EPS density in the saturated state was taken as 100 kg/m$^3$ both for EPS15 and EPS20.

The choice of EPS20 only for the top layer of the sub-base and EPS15 in the remaining layers was motivated exclusively by economical reasons. The price of the lighter EPS types is lower than that of the heavier EPS types. Material costs were thus reduced by building-in the lighter EPS15 in the lower sub-base layers. EPS15 is the lightest available EPS for road construction on the market. Furthermore, Blast Furnace Slags (BFS) were chosen for the unbound roadbase above EPS. Selection of BFS was based on the expectation of a relatively high elasticity modulus and its self-cementing properties which cause an increasing E-value in time.

The originally designed pavement structure, however, was revised to accomplish the second major criterion: the limited budget. Total costs were additionally cut down by reduction of the total EPS thickness, 0.8 m in stead of 0.8 m, in an effort to stay within the available budget. The resulting smaller stress reduction $\Delta \sigma_{ss}$ in the subsoil under the revised pavement structure was accepted as unavoidable. Weight-balance calculations and the resulting stresses in the subsoil ($\sigma_{so}$) under the revised concrete block pavement structure before and after reconstruction are shown in Figure 7.5.
The finally accepted pavement structure means that the occurrence of the ongoing settlements described in the subsection 7.3.3 were accepted as tolerable in return for achieved savings. The intention was more to design a settlement-poor block pavement structure than a settlement-free pavement structure. The use of a 0.5 m thick EPS layer in the sub-base enabled, however, the street level to be raised without increasing the dead weight. A thicker EPS layer would lead to reduction of the dead weight but economic reasons excluded this solution.

Use of a good quality unbound BFS material enabled the design of a thin roadbase at the first stage. Ultimately, the roadbase thickness was changed from 200 mm to 400 mm. The main reason for applying a thicker roadbase was to avoid a risk of severe rutting.

High safety factors are used in the stress calculations with respect to the dead weight of the EPS blocks. In Chapter 5 it was indicated that the weight of the immersed EPS20 blocks can be doubled when they stay under water for a long period of time. This implies that lower safety values may be used, i.e. 40 kg/m³ for EPS20 instead of 100 kg/m³. It is known from literature that the water absorption of EPS15 is somewhat higher than that of EPS20, so it is assumed that the maximum volumetric weight of EPS15 blocks is 40 kg/m³ as well. However, such lower safety values would lead to a very limited absolute change in stress reduction (0.6 kPa instead of 0.29 kPa).

Buoyancy forces have also to be considered in the scope of designing with EPS for a case of flooding of the areas where EPS fills are present. This might restrict the lower level of the EPS layer in order to keep the sub-base in position. For the buoyancy calculations low EPS densities (ρ_{EPS15} = 15 kg/m³ and ρ_{EPS20} = 20 kg/m³) and the highest
found groundwater level (NAP-1.84 m) were used. The Safety Factor (SF) obtained in [eq. 7.2] was higher than 1.1, the minimum value required in the literature [10].

\[
SF = \frac{G_{CS}^m G_{sand}^m G_{BFS}^m G_{EPS}^m g}{F_{buoyancy}} \approx (0.08 \times 2.4 + 0.1 \times 1.6 + 0.4 \times 2.2 + 0.2 \times 0.2 + 0.3 \times 0.015) \times g = 1.8 \quad \text{[eq. 7.2]}
\]

Where:  
SF - safety factor against buoyancy [\(\times\)]  
G, - weight of (dry) pavement layer \(i\) per m\(^2\) [kg]  
g - gravity [m/s\(^2\)]  
F - buoyancy force [N]

The details of the originally designed and of the final pavement structures are shown again in Figure 7.6. 80 mm rectangular concrete paving blocks from already existing pavements were laid over a 100 mm thick bedding sand layer. The BFS roadbase had a thickness of 400 mm. The sub-base consisted of two EPS layers: an upper 200 mm thick layer of EPS20 and the lower 300 mm thick layer of EPS15.

![Figure 7.6](image)

**Figure 7.6**  
Final design and original design of the concrete block pavement structure with EPS sub-base at Stolwijk

7.3.5 Conditions of the Pavement Structures

The visual inspection, performed in May 1993, pointed out that in general the considered pavement structures were in good conditions. The Rut Depth (RD) varied mostly between 5 and 10 mm, except for a few weak locations where these values were exceeded. It is recalled that a RD of 15 mm is generally used as a design criterion for concrete block pavement structures [11, 12]. In the following paragraphs observations during the inspections, made on the three considered streets (Korenbloemstraat, Dahiistraat and Bilwijkerweg) are presented.

No rutting was observed on the northwestern, wide part of the Korenbloemstraat while the maximum measured joint width amounted to 4 mm on the considered street part. On the next narrow part of the street RD ranged between 5 and 15 mm in the wheel track. Traffic situations differ on those two parts of the Korenbloemstraat with respect
to free space in the cross section available for traffic. Existing garages and wide parking places lead to the absence of parked cars on the northwestern street part. This enables the vehicles coming from opposite directions to pass each other without problems. On the second part the parked cars on both sides of the street leave a minimum space in the cross-section centre. The result is that all vehicles, from both directions, drive through the narrow part remaining in the middle of the road. In such a way, despite a low traffic intensity in the area, the passing of all vehicles over the same two wheel tracks caused substantial rutting on that part of the street.

On the subject of local unevenness, the drains built-in under the kerb stones settled more than the surrounding pavement surface. The depth of this local unevenness was sometimes larger than 30 mm, which is rated as a "serious unevenness" [7]. The explanation should be found in the adaption of the EPS thickness on these places in order to create enough space for building-in the drainage system. Consequently, the extra dead weight obtained in this way leads to larger local settlements than in the surrounding area.

Rut depths measured on the Dahliistraat amounted to 5 mm or less.

Regarding the Bilwijkersweg, rut depths measured on the northeastern wheel track along the canal were generally greater than the RD's on the inner wheel track. The weakest section of the road is at the southeastern end of the road where the largest FWD-deflections were always measured, as well as the maximum RD's of 25 mm (outer wheel track) and 13 mm (inner track). A few broken concrete blocks completed the picture of this location as an extremely weak one. Immediately on the next FWD measuring point no rut was observed. The reason for the obviously increased bearing capacity should be sought in the presence of kerb stones (0.15×0.2×1.0 m) on the roadside along the canal.

At the southeastern end of the road the pavement structure is not laterally supported. There are only thin profiled concrete tiles (0.6×0.4×0.08 m), with gaps for grass, driven into the verge along the pavement's edge. Their function is not to support laterally the concrete block toplayer but to protect the verge surface along the road edge. On this part of the road I-shaped concrete blocks were used in an attempt to strengthen the pavement structure in the transverse direction. However, the positive effect of such a measure is negligible if the roadverge does not provide a proper support.

No RD of importance was observed in front of house number 61, only a few meters beyond the weak point at number 51. At this location, kerb stones and a footway pavement are present on both sides of the road. Joint widths are less than 8 mm. In the northern continuation of the Bilwijkersweg, RD's have values between 5 mm and 8 mm.

From number 93 there is again a road section with driven-in thin profiled concrete tiles at the canal side. The local RD on the outer wheel track amounted to 23 mm on the street corner near number 93, while the joint widths were from 8 mm up to 12 mm. Values of RD's in front of number 95 were equal to 5 mm and then increased again to 20 mm (outer wheel track along the canal) at number 105. The situation of this section is similar to the second part of the Korenbloemstraat with respect to the available parking places so that the vehicles from both directions pass over the same wheel tracks. These observations are illustrated in Figure 7.7.
All in all, the three streets were generally spoken in fairly good condition. Only one very weak location was observed at the southeastern end of the Bilwijkerweg. This location, surrounded by water on two sides, was behind the bridge. An explanation for the bad pavement behaviour should be sought in a combination of two reasons: an inappropriate lateral support, if any, by thin driven-in profiled concrete tiles to the concrete element toplayer, on one hand, and a very poor bearing capacity of the subsoil at the considered place, on the other. In general, a weaker behaviour of the pavement structure was observed on the sections of the Bilwijkerweg without kerb stones (see Figure 7.7). It was manifested by larger deflections occurring on these road sections of the Bilwijkerweg, as can be seen in Figures 7.9 and 7.10 (corresp. house numbers are shown in Figure 7.7). The solution is replacement of the profiled concrete tiles with a kerb stone type prescribed in [13].

7.4 FWD RESULTS AND PAVEMENT ANALYSIS

7.4.1 FWD measurement characteristics

The first series of FWD measurements at Stolwijk were made on March 26, 1990 on the old concrete block pavement structures with a sand sub-base. The highest deflection under the centre of the loading plate amounted to about 1.8 mm. The deflections were due to a 50 kN load which is usually applied for such measurements. The second FWD measurements were carried out a few days after the reconstruction, therefore those measurements can be considered to be representative for the zero state. The applied force was again equal to 50 kN. Unfortunately, it was realized later that the registered deflections were out of the range (approx. 2 mm) of the geophones. On the basis of the results it was obvious that the surface deflections were very high but the obtained values could be biased. In the following series of FWD measurements, a 25 kN force was therefore applied. Even in this case the deflections were close to 2 mm.
Measurements on Block Pavement Structures

The regular FWD measurements (25 kN load) at Stolwijk were repeated about every three months, starting on December 17, 1990 till February 9, 1993; they represent 8 series of measurements. The measurement procedure consisted of applying one small initial load pulse, to settle the plate, followed by two load pulses of 25 kN. The deflections due to the second load pulse were registered. These FWD measurements provided data for analysis of the block pavement behaviour. The days when the measurements were carried out are presented in Table 7.1.

<table>
<thead>
<tr>
<th>Falling Weight Deflection Measurements at Stolwijk - block pavement structures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data</td>
</tr>
<tr>
<td>26-03*</td>
</tr>
</tbody>
</table>

* FWD measurements performed using 50 kN force

Table 7.1 - Days of FWD measurements on concrete block pavements in Stolwijk

The deflections were always measured at the same 39 locations per series. The measurement locations were fixed by house numbers. On the pavement of the Korenbloemstraat and the Dahliastraat deflection measurements were made at 10 and 9 points respectively. On the Bilwijkerweg measurements were carried out at 10 points in the right hand wheel track, in both directions. For illustration all the deflections measured during the very first measuring session performed before the reconstruction (March 1990) with a 50 kN load are shown in Figure 7.8. Furthermore the deflections measured during the first (December 1990) and the last session (February 1993) with a 25 kN load are presented in Figure 7.9 and 7.10 respectively. The measured deflections on the block pavement structure with an EPS sub-base were much higher than the deflections on the block pavements with a sand sub-base.

Figure 7.8 - Complete FWD deflections, measured in March 1990, due to a 50 kN force
Measurements on Block Pavement Structures

Figure 7.9 - Complete FWD deflections, measured in December 1990, due to a 25 kN force

Figure 7.10 - Complete FWD deflections measured in February 1993, due to a 25 kN force

7.4.2 FWD measurement results

The initial data are classified into four groups. One group is formed by the deflections on the Korenbloemstraat, the second group are the deflections on the Dahliastraat, the third consists of the measurements on the northeastern lane of the Bilwijkeweg and the last group on the southwestern lane of the Bilwijkeweg. For each group two representative points were chosen. Only the deflections at these representative points were used afterwards in the back-calculation analyses. In this section it will be described how these representative points have been determined.
As a first step all measured deflections were corrected to correspond to the 25 kN force. This is done by multiplication of the measurement values with the ratio between 25 kN and the registered force. The first series of measurements was an exception as the applied force is approximately 50 kN. In this case the deflections were multiplied by the ratio between 50 kN and the actual force value.

The second step was the determination of the measurement points which could be used as representative values for the whole group. This was carried out by selecting the points with a maximum deflection that came closest to the statistical 50% and 85% maximum deflection respectively, in the considered group. The criterion 50% was chosen in order to analyze the average conditions of the test sections. The 85% points were selected to gain insight into the condition on the locations where relatively high deflections were measured but to exclude eventual extremely weak points.

One has to realize that the deflection measurements were carried out 8 times with a load level of 25 kN. For each survey it was intended to select the 50% and 85% representative location using a statistical procedure. However this can only be achieved if the frequency distribution of the deflections could be described by some kind of distribution function. For that reason it was tried to fit so called β functions through the frequency distributions. These attempts were not successful, poor fits were obtained. A statistical analysis of the data by means of the programs MOMENTS and DATABELTA [14] pointed out the differences in coefficients of skewness and flatness, as well as shape factors of the analyzed \( d_{max} \) distributions (belonging to beta-distributions) between the measurements.

Then it was decided to determine the 50% and 85% locations by a manual procedure from the frequency distribution. Unfortunately not always the same representative locations were obtained. This means e.g. that the representative locations determined for the second deflection survey were different from the sixth and eighth survey. In Figures 7.11 (December 1990), 7.12 (June 1991) and 7.13 (February 1993) histograms of the maximum deflection for three measuring sessions are presented. In such conditions it was decided to select one single 50% and one single 85% representative location such that they were representative for all 8 surveys.

![Figure 7.11](image-url)

**Figure 7.11**

Frequency histograms of maximum deflections measured in December 1990
The location $d_{50}$ in the Korenbloemstraat was thus found in front of the house number 11 while the point $d_{85}$ was located in front of the number 19. In the case of the Dahliastraat these house numbers were 20 ($d_{50}$) and 6 ($d_{85}$). On the northeastern lane of the Bilwijkeweg the representative deflections were found in front of the house numbers 95 ($d_{50}$) and 105 ($d_{85}$) respectively. Furthermore, the point in front of 105 was selected on the opposite southwestern lane of this road to enable a comparison of the two points in the same cross section. An additional reason for the choice of this particular cross section was a large local rut depth of 20 mm. As the second selected point on the southeastern lane, the location in front of the number 69 was chosen. In the first place, its $d_{\text{max}}$ was close to $d_{50}$ and, in the second place, this is a location on the road section with a proper kerb, unlike the remaining three selected points on this road. These last two points are also notified as $d_{50}$ (69) and $d_{85}$ (105) in the further text.

The corrected deflections on the mentioned representative locations are shown in Figures 7.14 to 7.17. Each line represents the deflection registered by a certain geophone. In other words, the lines show the deflection at a certain distance from the loading centre. The appropriate distance is mentioned in the figures. The x-axis presents the period in months from the pavement reconstruction (in October 1990). The deflections are given on the y-axis in $\mu$m ($10^4$ m). The figures point out once more the very high values of the recorded deflections.
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Figure 7.14 - Representative deflections for 50% and 85% points resp. on the Korenbloemstraat

Figure 7.15 - Representative deflections for 50% and 85% points resp. on the Dahliastraat

Fig. 7.16 - Repr. deflections for 50% and 85% points resp. on the Bilwijkerweg-northeastern lane
Fig. 7.17: Repr. deflections for 50 % and 85 % points resp. on the Bilwijkerweg-southwestern lane

Important information on the performance of the concrete block pavements is the trend of the deflections in time. In case of stable concrete block pavement structures the deflection values decrease in time. This means that the pavement structures show the characteristic progressive stiffening behaviour under the traffic volumes. Data with respect to traffic loadings at Stolwijk point to a low traffic loading, especially on the Korenbloemstraat and the Dahliastraat. This implies that the stiffening is expected to be very slow.

An example of the linear relationship between the maximum deflection \( d_0 \) under the loading plate and the logarithm of the number of standard axle load repetitions \( \log N \) is given in the literature [12]. Following this relation, shown in Fig. 7.18, \( d_0 \) decreases with \( \log N \) more rapidly for pavement structures with a thinner sand layer. From Figure 7.18 it is clear that the measurements performed in Delft showed a decrease of deflection in time as well.

The analyzed block pavement structures at Stolwijk have a 0.5 m thick roadbase (0.1 m sand + 0.4 m blast furnace slag) above an EPS sub-base and the peat subsoil. Because of the high maximum deflections measured on the Stolwijk pavements it is assumed that the considered pavement structures correspond more or less to block pavement structures with a thin sand sub-base. Therefore the expectations were: high initial deflections \( d_0 \), and a rapid decrease of \( d_0 \) in time. The first expectation was right. The measured deflections on the reconstructed pavement structures have very high values. The second expectation is, at least partly, realized.
the Figures 7.14 to 7.17 a decrease of the deflection $d_\phi$ measured by geophone 1 can be noticed.

The observed decrease of the maximum deflection $d_\phi$ was not continuous. A temporary increase of the deflections occurred twice during the period the measurements were performed. The analysis of the Figures 7.14 to 7.17 pointed out that the reversed trend seems to be a seasonal phenomenon occurring in the autumn.

In order to find an explanation for the periodical increase of the measured deflections, meteorological data was analyzed for the considered area because it is known that there are seasonal influences on the pavement's behaviour [15]. Especially precipitation and evaporation data are of interest since it is shown [16] that seasonal variations in deflection levels correlate with seasonal variations in precipitation and evaporation. Precipitation data are collected in Gouda, a city very close to the test area, while data about evaporation are available for Rotterdam airport, some 23 km away from Stolwijk. The precipitation and evaporation for the considered period are shown in Figure 7.19. Simultaneously the difference between the total precipitation and evaporation is presented.

The correlation between deflections and net precipitation becomes really obvious when the cumulative net precipitation curve is shown in one graph with the deflections; this is done in Figure 7.20. In this figure the $d_{\text{max}}$ (50%) curve presents the average values of the deformations in the loading centre (geophone 1) from all four $d_{\phi}$ selected points. The same is valid for the curve $d_{\text{max}}$ (85%). Trends of all three curves are similar. During the period of a precipitation surplus, the moisture content in the unbound layers will increase which results in an increase of the deflections; this happens during the autumn. During the summer, when there is a precipitation deficit, the unbound materials will have a lower moisture content, consequently a higher stiffness, which in turn results in lower deflections.

![Figure 7.19 - Precipitation and evaporation in the regions Gouda and Rotterdam during the period when FWD measurements were performed](image-url)
On a few locations the measured deflections had up to two times higher values than the mean deflections on the same street. The visual inspection pointed out that among these 'weak cross sections' there were sections with drains on both edges of the road. Large local settlements were also noticed at the places with drains. The explanation is a partial removing of EPS in order to make free space for construction of the drains. This replacement of EPS means extra dead weight which stimulates settlements. These cross-sections where EPS was partly removed, should be considered as critical locations. Accordingly, additional attention should be paid during the design and construction (compaction aspects) of such cross-sections to avoid local problems on block pavement structures with an EPS sub-base.

### 7.4.3 Back-calculation analysis procedure

The back-calculation analysis started with modelling the considered pavement structure. In the selected pavement model, the layers are assumed to be constant in thickness and the materials are assumed to be linear elastic. These assumptions allow the use of linear-elastic multi-layer programs (e.g. BISAR) for the calculation of deflection bowls. By means of iterative calculations the elasticity moduli of the layers are determined such that the calculated deflection bowls match as close as possible the measured ones.

The analyzed pavement structure is shown in Figure 7.21. The needed values for Poisson's ratio, $\nu$, in the model have been taken from the literature [18]. The number of layers is dictated by the number of geophones which limits the maximal number of layers in the model. The maximum number of model layers is equal to the number of geophones minus one. Consequently, one 0.5 m thick EPS layer was introduced instead of one layer of 0.2 m EPS20 and one layer of 0.3 m EPS15 respectively.
The back-calculation procedure was repeated for the corresponding measurements for all four groups of data and for both the $d_{30}$ and $d_{35}$ deflection bowls. With 'corresponding measurement' those measurements were indicated which had been performed since December 1990. All of them were done using an identical force value (25 kN) and loading procedure (the second load pulse is relevant). The differences between the corrected measured deflections and the deflections computed by BISAR were almost always within 5%.

The deflection measured at a distance of 0.3 m from the loading centre was not compared with the calculated values. At such a close distance the measured deflections were mostly lower than those calculated. These deflections reflect the behaviour of the top layer. In the case of a concrete block pavement structure, this layer consists of blocks and joints between them, filled with sand. The blocks directly hit by the falling weight, move partly downwards along the joints. The adjacent blocks are only pulled down to a limited extent because of the limited load transfer in the joints. This 'shear force' behaviour of the concrete block layer cannot be simulated by means of a linear-elastic layer approach. The difference between the shear toplayer in reality and the bending layer in the BISAR model results in a higher calculated $d_{3,3}$ deflection than actually occurs. The described phenomenon, particularly occurring with large deflections, is illustrated in Figure 7.22.

Deflections at a greater distance reflect the deformations occurring in the lower pavement layers. Such layers behave much more like a homogeneous and isotropic layer than the concrete block toplayer.
7.4.4 Back-calculated layer E-moduli

The results of the back-calculation are presented in Tables 7.2 to 7.5. The first column ‘Age’ represents the period between the reconstruction of the block pavement structures and the FWD measurement in question. In other words, it is the period (in months) between October 1, 1990 for Korenbloemstraat and Dahliastraat, or October 20, 1990 for Bilwijkeweg, and the dates listed in Table 7.1. The grey columns give the back-calculated E-moduli of the different layers in MPa. In the columns ‘DIFF’ the largest difference between a measured and a back-calculated deflection value is listed.

<table>
<thead>
<tr>
<th>Age [month]</th>
<th>CB</th>
<th>sand</th>
<th>BFS</th>
<th>EPS</th>
<th>subsoil</th>
<th>DIFF back-calculated E modulus</th>
<th>[MPa]</th>
<th>DIFF back-calculated E modulus</th>
<th>[MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dec'90</td>
<td>2.6</td>
<td>200</td>
<td>35</td>
<td>55</td>
<td>12</td>
<td>21.8</td>
<td>5.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jun'91</td>
<td>8.3</td>
<td>1700</td>
<td>51</td>
<td>185</td>
<td>11</td>
<td>23</td>
<td>3.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oct'91</td>
<td>12.1</td>
<td>1400</td>
<td>40</td>
<td>190</td>
<td>4.9</td>
<td>35.5</td>
<td>3.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan'92</td>
<td>15.2</td>
<td>1500</td>
<td>35</td>
<td>125</td>
<td>6.1</td>
<td>25</td>
<td>4.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apr'92</td>
<td>18.9</td>
<td>1700</td>
<td>65</td>
<td>220</td>
<td>6.2</td>
<td>23</td>
<td>1.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aug'92</td>
<td>22.2</td>
<td>2000</td>
<td>80</td>
<td>280</td>
<td>6</td>
<td>21.5</td>
<td>1.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nov'92</td>
<td>25.2</td>
<td>2700</td>
<td>65</td>
<td>275</td>
<td>4.2</td>
<td>31.5</td>
<td>3.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb'93</td>
<td>28.3</td>
<td>2700</td>
<td>63</td>
<td>325</td>
<td>5</td>
<td>26</td>
<td>3.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

DIFF - maximum difference between a back-calculated and measured deflection (excl. geophone 0.3 m)

**Table 7.2** - Back-calculated E moduli of the layer materials in concrete block pavement structures at Stolwijk, based on FWD results on representative measuring points

<table>
<thead>
<tr>
<th>Age [month]</th>
<th>CB</th>
<th>sand</th>
<th>BFS</th>
<th>EPS</th>
<th>subsoil</th>
<th>DIFF back-calculated E modulus</th>
<th>[MPa]</th>
<th>DIFF back-calculated E modulus</th>
<th>[MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dec'90</td>
<td>2.5</td>
<td>1100</td>
<td>60</td>
<td>40</td>
<td>8.1</td>
<td>25.5</td>
<td>4.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jun'91</td>
<td>8.3</td>
<td>1500</td>
<td>55</td>
<td>95</td>
<td>10.5</td>
<td>21.5</td>
<td>4.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oct'91</td>
<td>12.1</td>
<td>1600</td>
<td>100</td>
<td>110</td>
<td>5</td>
<td>48</td>
<td>2.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan'92</td>
<td>15.2</td>
<td>1900</td>
<td>40</td>
<td>70</td>
<td>17</td>
<td>16.8</td>
<td>9.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Apr'92</td>
<td>18.9</td>
<td>1600</td>
<td>50</td>
<td>157</td>
<td>6</td>
<td>29</td>
<td>3.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aug'92</td>
<td>22.2</td>
<td>2100</td>
<td>40</td>
<td>220</td>
<td>10.5</td>
<td>22.7</td>
<td>4.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nov'92</td>
<td>25.2</td>
<td>2700</td>
<td>85</td>
<td>266</td>
<td>5</td>
<td>35</td>
<td>2.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb'93</td>
<td>28.3</td>
<td>2400</td>
<td>30</td>
<td>200</td>
<td>5.9</td>
<td>28</td>
<td>3.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

DIFF - maximum difference between a back-calculated and measured deflection (excl. geophone 0.3 m)

**Table 7.3** - Back-calculated E moduli of the layer materials in concrete block pavement structures at Stolwijk, based on FWD results on representative measuring points
Measurements on Block Pavement Structures

<table>
<thead>
<tr>
<th>Age [month]</th>
<th>Bilwijkweg 95 northeastern lane - 50%</th>
<th>Bilwijkweg 105 northeastern lane - 85%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CB sand BFS EPS subsoil DIFF</td>
<td>back-calculated E modulus [MPa] [%]</td>
</tr>
<tr>
<td>Dec’90</td>
<td>1.9 1300 100 24 5.4 26.5 4.3</td>
<td>1100 90 27 3.9 44 3.1</td>
</tr>
<tr>
<td>Jun’91</td>
<td>7.7 2400 70 19 7.1 18.9 4.5</td>
<td>1700 120 21 4.2 28.5 2.3</td>
</tr>
<tr>
<td>Oct’91</td>
<td>11.5 2400 90 38 4.5 31 2.3</td>
<td>2000 40 37 5.2 22.5 2.7</td>
</tr>
<tr>
<td>Jan’92</td>
<td>14.8 2500 140 36 4.9 25 2.7</td>
<td>2200 60 15 7.5 16 4.0</td>
</tr>
<tr>
<td>Apr’92</td>
<td>18.3 2500 120 36 5 23 2.0</td>
<td>2000 55 20 4.8 17 0.9</td>
</tr>
<tr>
<td>Aug’92</td>
<td>21.5 2900 160 69 6.8 24 1.4</td>
<td>2700 80 42 4.2 20 1.7</td>
</tr>
<tr>
<td>Nov’92</td>
<td>24.6 3500 190 59 5.1 37.6 3.2</td>
<td>3000 55 42 4.9 21 1.7</td>
</tr>
<tr>
<td>Feb’93</td>
<td>27.7 3400 150 35 4.7 27.5 0.9</td>
<td>3000 30 35 4.5 19 1.1</td>
</tr>
</tbody>
</table>

DIFF - maximum difference between a back-calculated and measured deflection (excl. geophone 0.3 m)

Table 7.4 - Back-calculated E moduli of the layer materials in concrete block pavement structures at Stolwijk, based on FWD results on representative measuring points.

<table>
<thead>
<tr>
<th>Age [month]</th>
<th>Bilwijkweg 69 southwestern lane - 50%</th>
<th>Bilwijkweg 105 southwestern lane 85%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CB sand BFS EPS subsoil DIFF</td>
<td>back-calculated E modulus [MPa] [%]</td>
</tr>
<tr>
<td>Dec’90</td>
<td>1.9 1900 40 31 5.3 29 2.8</td>
<td>2100 45 32 4.4 43 3.9</td>
</tr>
<tr>
<td>Jun’91</td>
<td>7.7 3300 220 72 9 22 0.5</td>
<td>3100 95 48 5.4 25 2.2</td>
</tr>
<tr>
<td>Oct’91</td>
<td>11.5 2300 60 115 4.3 44 4.0</td>
<td>3700 210 68 4.3 33 1.9</td>
</tr>
<tr>
<td>Jan’92</td>
<td>14.8 3700 100 57 6.8 22.5 4.1</td>
<td>3000 65 53 5 29.5 2.2</td>
</tr>
<tr>
<td>Apr’92</td>
<td>18.3 3200 75 150 5.2 29 0.9</td>
<td>3000 90 80 4.6 22.5 2.4</td>
</tr>
<tr>
<td>Aug’92</td>
<td>21.5 3500 67 290 6.3 24 1.7</td>
<td>3700 185 135 5 20 2.7</td>
</tr>
<tr>
<td>Nov’92</td>
<td>24.6 3800 61 274 5.1 28.5 1.7</td>
<td>3900 91 135 7 18 2.7</td>
</tr>
<tr>
<td>Feb’93</td>
<td>27.7 3500 70 80 4 36.5 1.0</td>
<td>3700 78 100 4.2 25 2.2</td>
</tr>
</tbody>
</table>

DIFF - maximum difference between a back-calculated and measured deflection (excl. geophone 0.3 m)

Table 7.5 - Back-calculated E moduli of the layer materials in concrete block pavement structures at Stolwijk, based on FWD results on representative measuring points.

The back-calculated E-modulus for EPS has values between 4 and 12 MPa. This is in fairly good agreement with the E-values which are normally reported. Also they correspond well with the values reported in Chapter 5 of this study.

As expected both the back-calculated E values for the roadbase and the moduli in the toplayer tend to be higher for the d_{50} deflection bowls than those obtained for the d_{85} bowls.

The elasticity moduli back-calculated for Blast Furnace Slags (BFS) were very low compared to the values mentioned for this material in literature [9], which amount to bet-
Measurements on Block Pavement Structures

ween 400 and 800 MPa. Furthermore, BFS is expected to have self-cementing properties which result in an increasing stiffness in time to about 1,000 MPa. A very slight increase in the E modulus of BFS can be recognized in the analysis results but the back-calculated moduli were only a fraction of the expected values and apparently no binding was realized. The reason for this is twofold:

a) because of the high deflection, indicating a significant amount of movement in the structure, binding cannot take place;

b) EPS, a material with an E-modulus of about 5 MPa, hardly provides any support to the roadbase. Absence of the appropriate bottom support influences the creation of all-round confining stresses in the roadbase. Because this confinement will hardly develop, any unbound base material cannot develop a significant stiffness.

Therefore, it is questionable whether the use of such a relatively expensive ‘self-cementing’ material is adequate above the EPS sub-base if the effective BFS stiffness does not exceed that of cheaper materials.

Tables 7.4 and 7.5 point out how the difference in lateral support from the roadedge also influences the $E_{BFS}$ moduli in the roadbase. For the northeastern lane along the canal (see Figure 7.7) lower $E_{BFS}$ values were back-calculated than on the southwestern lane of the Bilwijkerweg. One should pay attention to the values regarding house number 105 where deflection bowls on both lanes were analyzed. The E-values in the top layer are, likewise, somewhat lower on the lane closer to the canal.

The highest E moduli for the BFS roadbase in the Bilwijkerweg were back-calculated on the southwestern lane on the part with proper kerb stones (point 69). This finding, generally lower deflections measured on this part of the road (see Fig.7.9 and 7.10) and no ruts on the considered road section (see Fig.7.7), pointed out the necessity of applying kerb stones which are able to give confinement. This is a warning for a designer. The improper lateral support of concrete block pavement structures with the EPS sub-base, such as on parts of the Bilwijkerweg with thin profiled concrete tiles driven-in along the road edge, leads to a weaker behaviour of the whole pavement structure, including the deep laid roadbase. The consequence is the occurrence of severe rutting in the wheel tracks in spite of the low traffic volume.

The more intensive traffic on the Bilwijkerweg compared to the other two measuring sections reflects on the stiffness of the top layer. Back-calculated E-values for the top layer on this road were larger than those on the Korenbloemstraat and the Dahliastraat which are subjected to a lower traffic loading. Consequently, one can still expect a certain stiffening of the concrete block layer on the considered streets with time.
7.5 CONCLUDING REMARKS ON CONCRETE BLOCK PAVEMENTS

- The measured deflections on concrete block pavement structures with an EPS sub-base at Stolwijk were much higher than the deflections on classic block pavements with a sand sub-base. This can be explained by the lower E-modulus of the used EPS compared with a sand sub-base.

- In general the condition of the block pavement structures at Stolwijk could be rated as rather good after being 3 years in service. On all sections the deflections decreased in time which is the characteristic progressive stiffening behaviour for stable block pavements.

- On a few locations the deflections are two times higher than the average deflections on the same street. These critical positions are often located in the cross-sections with drains on both sides of the road. Also, large local settlements were noticed at these locations. There the local dead weight of the pavement structure is higher because part of the EPS layer is replaced by the drains. All this means that cross-sections with drains should be carefully designed and constructed (compaction aspects) in order to avoid the local problems on the block pavement structures with an EPS sub-base.

- A modulus of elasticity of 5 MPa, which is usually assumed for EPS15 and EPS20, seems to be a proper value for the design of block pavement structures. The back-calculated $E_{\text{EPS}}$ had values between 4 and 12 MPa which means that an E-value of 5 MPa should be safe enough for use in practice.

- The results of the back-calculation analyses performed on $d_{50}$ and $d_{95}$ deflection bowls pointed out a great influence of the effective stiffness of the unbound material on the general pavement behaviour. It also emphasizes the importance of a proper compaction of the unbound roadbase materials in structures with an EPS layer.

- The back-calculated elasticity moduli for the Blast Furnace Slags (BFS) roadbase amounted to less than 50% of the potential values for this material. The expected self-cementing apparently did not occur in the BFS at all. Therefore, the choice of the relatively expensive BFS should be critically reviewed because in this case its effective stiffness does not exceed that of cheaper materials.

- Driving-in thin profiled concrete tiles in a weak road verge is not an adequate substitute for heavy kerb stones. This is because the thin concrete tiles cannot provide the needed lateral support to concrete block pavement structures. Insufficient lateral support not only results in wide joints and ineffective load transfer between the blocks but also to a certain decrease in the effective stiffness of the lower built-in roadbase layer. The final result is the rapid occurrence of severe ruts in the wheel track on sections where the concrete tiles have been used.
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CHAPTER 8

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8.1 INTRODUCTION

In this chapter surface deflection measurements and asphalt strain measurements are described that were performed on the Matlingeweg in Rotterdam. It was interesting to investigate this road because of its sub-base, which consists of a 1.0 m thick EPS layer, and its heavy traffic loading. The measurements were carried out by means of the Falling Weight Deflectometer (FWD) and strain transducers respectively. Four transducers were built-in at the bottom of the asphalt layer for this purpose.

In Section 8.2 the measuring program is discussed. Firstly, the FWD measurements are described that are performed to enable the back-calculation of the modulus of elasticity of the individual pavement layers. The layer moduli give an impression about the actual bearing capacity of the pavement structure. Secondly, the same is done for the measurements of the tensile strain in the asphalt layer. The maximum value of the horizontal tensile strain at the bottom of the asphalt layer is a dominant criterion in asphalt pavement design. The value of the asphalt strain increases in time due to the damaging effect of the traffic loading. Accordingly, information about the development of the horizontal asphalt strain in a real pavement structure permits the analysis of its structural behaviour. Details about the strain transducers which were used in this investigation complete this section.

Section 8.3 gives more details about the measuring site. Information about the measurement location and traffic is given in the first subsection. The Matlingeweg, the road where the measurements were performed, is a connecting road between two heavily trafficked motorways and crosses an industrial estate under construction. Accordingly, a large amount of heavy trucks uses the Matlingeweg. Furthermore the criteria are analyzed for the choice of an EPS sub-base on two sections of the Matlingeweg. Observed local settlements which occurred before the road reconstruction (when the EPS was built-in) are described. The results pointed to a varying subsoil bearing capacity along the road alignment. EPS was used only on the sections where extreme settlements were observed in the past. Further in this section, information is given about pavement layer materials and thicknesses. It is also described that overlaying the pavement structure has taken place much earlier than was originally planned because severe cracking occurred at the pavement's surface within a few weeks after reconstruction.

FWD measurement results are listed in Section 8.4. Interpretation of the results was complicated by different temperatures during the measurements. Asphalitic concrete layers at the top contain visco-elastic bitumen, which has a temperature dependent behaviour, as a binder. Practical consequences are that the FWD deflections and the horizontal asphalt strains increase with increasing temperature. Back-calculation of the layers moduli is performed and the back-calculated moduli were used in the pavement analysis.

Asphalt strain measurement results are discussed in Section 8.5. The temperature-dependent behaviour of the asphalt layer prevents a direct comparison between the measured strain values. It implies that these values have to be transformed to a reference
temperature before comparison. The way this was done is described in this section. The back-calculated E-values in the pavement structure layers were used for this purpose. Finally, the trend of the translated asphalt strain values (at the reference temperature of 20°C) is presented as a function of the pavement age.

In Section 8.6 the general conclusions on measurement and calculation results are drawn. They refer to the pavement condition in general (after 3 years in service) and the elasticity moduli of the pavement layers in particular.

8.2 MEASURING PROGRAM

In order to get an insight into the structural behaviour of flexible pavement structures with an EPS sub-base, four asphalt strain transducers were built-in at the bottom of the Matlingeweg in Rotterdam. By means of these transducers the asphalt strains under FWD loading have been measured about three times a year from October 1990 (completion of the pavement reconstruction, i.e. opening to traffic) to June 1993. Simultaneously, FWD deflection curves have been measured.

8.2.1 FWD measurements

As mentioned already in Chapter 6, measuring the surface vertical displacements (deflections) due to a dynamic loading is a non-destructive method for the evaluation of the structural condition of pavements. The deflection bowl reflects the manner of load spreading in the pavement structure. Deflection measurements provide necessary input data for the back-calculation of the modulus of elasticity of the individual pavement layers. The back-calculated layer moduli give an impression about the actual bearing capacity of the pavement structure. The deflections caused by the load were registered by six geophones placed from the centre of the loading plate up to a distance of 1.8 m from the plate centre. The complete back-calculation results are described in subsection 8.4.2 of this thesis.

8.2.2 Asphalt strain measurements

The asphalt strain criterion is the most important of two criteria normally used for the design of flexible pavement structures. The second criterion is the vertical compressive strain at top of the subgrade. The design model for pavement structures is a linear-elastic multi-layer system (shown in Figure 8.1) where the materials are considered to be linear-elastic, homogeneous and isotropic [1]. Calculation of the horizontal tensile asphalt strain is carried out using the linear-elastic multi-layer computer program BISAR. The design life is expressed in ‘allowable number of 100 kN standard axle load repetitions’. This number is, in most cases, determined from relationships between asphalt strain and allowable strain repetitions. All this emphasizes the importance of knowing the tensile
strain value at the bottom of the asphalt layer for an appropriate estimation of the actual pavement condition.

Four strain transducers (Figure 8.2) were laid on the roadway during the road construction of the Matlingeweg in October 1990 just before paving with asphalt. The transducers were built-in on the northwestern part of the Matlingeweg where the EPS sub-base is 1.0 m thick. The position of the strain transducers in the cross-section of the road, at a distance of 1.4 m from the pavement edge, was chosen such that the transducers lie in the centre of the wheel track. This is the place in the road cross-section where the vehicles cause the biggest damage to the asphalt. Accordingly, the effect of traffic loading in time is first noticeable in these places.

The distance between the transducers in the longitudinal direction was designed to correspond with the radial offsets of the geophones from the loading plate of the FWD equipment. This was only valid for the first three transducers (no. 837, 838 and 839). The last transducer, no. 840, was placed at a distance of 4.8 m from the first one in order to create an input data for a finite element model with a radius of 5.0 m. Four transducers were built-in to have enough operational measuring instruments in the case of a defect in one or two of them. The roadrollers had to drive above the transducers during the asphalt paving, which could have damaged these instruments. Fortunately, all four transducers are still in operation. Asphalt strain measurement results are presented in subsection 8.5.

8.2.3 Measurement procedure

The first FWD measurements on the Matlingeweg have been done on October 31, 1990, a few days after reconstruction. These measurements could therefore be considered to
be representative for the zero state. Loading of the pavement was exerted on a 300 mm diameter plate by a falling weight, part of the FWD equipment. The exact value of the exerted force was measured for each loading. The measured deflections and strains were then normalized to a 50 kN force corresponding to a 100 kN standard axle load.

The second measurements were carried out before overlaying the pavement structure in December 1990. The measurements from January 1991 till June 1993 were performed on the pavement structure with an extra 80 mm asphalt layer at the top. The applied force was again equal to 50 kN. In figures, this means 2 series of measurements on the pavement structure without overlay and 8 series of measurements on the structure with an asphalt overlay. These measurements provided data for analyses of the pavement behaviour in the period between October 1990 and June 1993. The dates when the measurements were done are presented in Table 8.1.

<table>
<thead>
<tr>
<th>Measurements on the Mattingeweg - flexible pavement structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>31-10* 13-12* 17-01 25-03 11-04 21-08 07-10 15-04 27-08 17-06</td>
</tr>
<tr>
<td>* before overlay</td>
</tr>
</tbody>
</table>

Table 8.1 - Measuring dates on flexible pavement structure of the Mattingeweg

The deflection measurements were always done at the same places, above the built-in strain transducers, marked with the numbers 837 ... 840 in Figure 8.3. A single measurement session consisted of five series; in each series the strain obtained in all four transducers during the FWD loading at one measurement point was registered. The FWD loading was exerted first at the point 838 (the first series), then at 839, at 840, then again at 838 and, finally, at 837 (the fifth series). The measurement locations were fixed by white rings pointed at the pavement's surface.

The measurement procedure was always the same with respect to the electronic equipment for registration of the signals and to the order in which the measurements were performed at the measurement points. A standard procedure makes it easier to compare the results obtained during measurements performed at different times. It avoids the effects of, for example, another set-up of the frequencies on the signal filters, or another position of the measurement vehicle with respect to measurement point etc., on the measured values.

The analyzed pavement structure was not equipped with built-in thermo-couples in the asphalt. The temperatures assumed for the pavement were measured at the bottom of a borehole in the asphalt, at a depth of about 50 mm. The hole depth is less than the bottom of the asphalt layer where the strain transducers were laid. The total asphalt thickness was about 130 mm before and about 210 mm after overlaying. Therefore, the temperature values measured in the borehole are an estimate of the asphalt temperatures.
In the cases when the temperature values ranged between 10°C and 20°C in the borehole, these have been downgraded by 1°C before they have been used as the asphalt temperature in pavement analysis. Furthermore, the asphalt temperature has been assumed to be 2°C lower than the borehole temperature when this exceeded 20°C. The correction values of 1°C and 2°C were determined on the basis of the temperature distribution in pavement structures with an EPS sub-base, described in Chapter 3 and the literature [2]. The part of the day when the FWD surveys were done, mostly between 10 a.m. and 3 p.m, was taken into account for the estimation of the difference between the temperatures at the bottom of the borehole and in the middle of the asphalt layer.

8.3 DESCRIPTION OF THE MEASURING SITE

8.3.1 Plan and traffic characteristics

The Matlingeweg is situated in the Northwestern part of Rotterdam. This road is an alternative route between the motorways A13 and A20 and it crosses the new industrial estate Rotterdam North-West. The position of the Matlingeweg between two heavily trafficked motorways is a reason for the large number of vehicles using this road. Another reason is its central location in the industrial estate under construction. Because of the very intensive construction activities in this area a larger than average number of very heavy trucks contributes to the heavy traffic on the Matlingeweg. The construction traffic for the new industrial estate has to use this road. The trucks used for transport of the building material (concrete elements, foundation piles etc.) usually have axle loads close to or even more than the maximum allowable values. The northern part of the Matlingeweg with the EPS sub-base lies just at the beginning of the industrial area, it is more or less the entrance to the area. Therefore, practically the entire traffic flow passes over the pavement test section which is drawn in Figure 8.4. The asphalt strain transducers are situated in the outer wheel track in the outer lane because this segment of the road cross section is used by all trucks and other heavy vehicles.

Figure 8.4

Plan of the northern part of the Matlingeweg with strain transducers

EPS sub-base + asphalt strain transducer

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8.3.2 Settlements

The part of the area, where the Matlingeweg is located, once was the bed of one of the branches of the river Meuse. Later, the original sediments were covered by waste material spoil (dredgings), deposited there during the long-term draining of the surrounding area. The existence of silt and spoil layers at certain depths in the subsoil implies its poor bearing capacity. Resulting settlements have values varying from a few mm up to 200 mm and more per year. The variation in settlements is caused by differences in local soil profiles. The upper layers consist of material that was brought in during previous road reconstructions and preparation of the site for building. Therefore the settlement values depend on the location. The Matlingeweg crosses the described area and the remarks about differential local settlements are valid for the various sections of this road. Sinking of the pavement surface is a few times larger on some sections than on others. The thickness of the EPS sub-base and the length of the various sections on the Matlingeweg were chosen on the basis of the observed local settlements.

8.3.3 EPS sub-base design

The flexible pavement structure of the Matlingeweg is partly laid on an EPS sub-base. Actually, EPS was built-in on two separate sections of this road, with lengths of about 175 m and 90 m respectively. A sand sub-base is applied in the remaining part of the Matlingeweg. Furthermore, the EPS sub-base has two different thicknesses on the sections where it was applied. The EPS layer is 0.5 m thick at the ends of the two above-mentioned sections and 1.0 m thick in the central part of both sections. In this way the quantity of EPS used was minimized for economic reasons.

The settlement values and corresponding pavement structure design are shown in Figure 8.5 for the longer of the two sections of the Matlingeweg with an EPS sub-base. A 0.5 m thick EPS layer was built-in on the terrain which had settled by about 0.5 m. A 1.0 m thick EPS package of two layers was laid where the total settlement reached more than 0.7 m during three years, and also where the pavement surface sank more than 0.3 m below the design road surface level. Larger settlement values indicated segments with a very low bearing capacity of the subsoil. The 0.5 m thick EPS layer was used to obtain a gradual change between the road parts on relatively weaker subsoil and the attached road parts on the subsoil with a somewhat higher bearing capacity.

EPS30 was used in the upper EPS layer and EPS25 in the second layer. EPS30 and EPS25 are expanded polystyrene foams with densities of 30 kg/m³ and 25 kg/m³ respectively. The most widely used EPS type in road construction is EPS20 which means that, in the case of the Matlingeweg, heavier types than usual were used. The advantages of EPS30 and EPS25 over EPS20 are a somewhat higher elasticity modulus and a slightly lower maximum level of water absorption. The strength and E-value of the EPS increase with density [3]. EPS20 has an E-modulus of about 5 MPa (under slow compressive loading), E-values for EPS25 and EPS30 amount to about 8 and 10 MPa respectively.
Measurements on a Flexible Pavement Structure

[4]. Relatively this a large difference in elastic modulus but absolutely this is only a small difference. The difference in water absorption level is not of practical importance. The disadvantage of heavier EPS types is that they are simply more costly. The question is whether a few MPa more in elasticity modulus in the sub-base justifies the accompanying extra costs. The larger the quantity of EPS applied, the more important this consideration becomes.

![Graph showing settlements and longitudinal profiles of the Matlingeweg](image)

**Figure 8.5** - Previously obtained settlements and longitudinal profiles of the Matlingeweg

### 8.3.4 Flexible Pavement Structure

The roadbase in the pavement structure of the Matlingeweg consists of a 0.4 m thick layer of a mixture of crushed masonry and crushed concrete, and a 0.15 m thick layer of sand above EPS. The applied roadbase material is an unbound material with a grain size distribution between 0 and 40 mm. This material is produced in a recycling process (using a stone-breaker) from bricks and concrete pieces that remain after demolition work. On the road sections without an EPS sub-base, the thickness of the crushed masonry/concrete base layer is 0.3 m.

The drain sand above the EPS layer(s) also protects the plastic sheet on the top of the EPS against mechanical damage. The plastic sheet also has to prevent leaked oil derivatives from reaching the EPS, as this could then be dissolved.
Measurements on a Flexible Pavement Structure

The asphalt package on the top was laid in two phases. Firstly, an open asphaltic concrete layer, type 0/22, with a thickness of about 80 mm, was laid over the roadbase. A top layer of 50 mm thick dense asphaltic concrete was laid during the road construction in October 1990. In December 1990, much earlier than planned, the pavement structure was overlaid by a 80 mm thick layer of dense asphaltic concrete. Figure 8.6 shows the pavement structure including the overlay on the Matlingeweg.

Figure 8.6: Pavement structure (incl. the asphalt overlay) with the EPS sub-base on the Matlingeweg

8.3.5 Damage on pavement structure and overlay

The original intention was to start the second phase of the project (i.e. an overlay) a year after reconstruction, when the works on the roads in the surrounding area would have been finished. The original design overlay thickness was 50 mm, and dense asphaltic concrete was chosen as the overlay material. However, damage on the northeastern lane of the Matlingeweg led to a complete change of plans. Alarming early failure was observed one month after the road reconstruction was finished and the road was opened to traffic. Cracking occurred in the longitudinal direction in the section with a 0.5m thick EPS sub-base at the location marked in Figure 8.7.

Figure 8.7: Location of the initial cracking in December 1990 on the Matlingeweg

Before the overlay was laid two observation pits were made to inspect the overall structure. One pit was made near the centre line between the two lanes (location 1) and
the other near the pavement edge (location 2). Figures 8.8 and 8.9 give a view of the condition at the joints in the EPS sub-base.

The EPS blocks were not properly laid. Instead of full contact between the blocks, which would enable the interaction between them, the blocks were laid with joints between them. Figure 8.8 shows that the width of the transverse joints between the EPS blocks was approximately 20 mm. Figure 8.9 gives an indication of the longitudinal joint. Although the joint was closed at this location a vertical height difference of approximately 5 mm could be observed. Furthermore, the unbound roadbase material appeared to contain many fine particles and was not well graded; it looked almost like a sand.
The reasons for the early deterioration were quite obvious in this case.

Firstly, the presence of open joints results in a total absence of any load transfer between the blocks.

Secondly, it was observed that the longitudinal joint between the blocks was located just in the middle of the two lanes. This is a very critical location since the heavy wheel loads are almost right above the joint. Because heavy trucks almost exclusively used the outer traffic lane the inner wheel track corresponds with the position of the longitudinal joints as schematically illustrated in Figure 8.10. Consequently, the heavy trucks loaded the pavement structure at those places where the unbound roadbase material had a strongly reduced support because of the open joints in the layer below. This led to a reduced support of the roadbase to the asphalt layers. An additional handicap of the pavement structure at the considered location was the presence of a single EPS layer. The visible result was the longitudinal cracking at the pavement’s surface.

Thirdly, high deflections were measured on the structure (see also section 8.4) which indicated that the crushed masonry/concrete base had a low stiffness and that high tensile strains were developing in the asphaltic layer.

**Figure 8.10**

Cross section of the Mattinge-wey with the inner wheel track corresponding to the position of the longitudinal joint between the EPS blocks.

From the analysis that was made it was concluded that most of the problems could be attributed to a less than optimal design (joint very close to the heavy loads), to a less than optimal construction (open joints) and to the fact that the structure as designed lacked sufficient stiffness. Therefore the overlay thickness was redesigned and increased from 50 to 80 mm in order to reduce the chance on the occurrence of cracks during the expected pavement design life.

The section where the cracking of the asphalt occurred was totally reconstructed. The asphalt layers were removed and the upper 150 mm of the roadbase were stabilized to improve its bearing capacity before overlaying with new asphalt layers with a total thickness of about 210 mm.

The pavement structure was not damaged on the road section where the strain transducers had been built-in. However, the road edge was quite seriously damaged about thirty meters further to the south.
8.4 FWD MEASUREMENT RESULTS AND PAVEMENT ANALYSIS

8.4.1 FWD measurement results

The temperature dependent behaviour of the asphalt layers has a big influence on the behaviour of the whole pavement structure. With respect to the FWD measurements the influence of temperature on pavement behaviour was expressed in lower measured deflections at lower temperatures on the Matlingeweg. A clear correlation between the measured deflection and temperature values can be seen in Figures 8.11 and 8.12 which show the measured deflections as a function of time for two measuring points on the Matlingeweg.

Figure 8.11
FWD measurement results above the strain transducer 838 on the Matlingeweg

Figure 8.12
FWD measurement results above the strain transducer 837 on the Matlingeweg
Measurements on a Flexible Pavement Structure

The FWD measurement results for the first two surveys are very different than the remaining ones because of the 80 mm thick asphalt overlay that was applied at the end of 1990. The measure in which this extra asphalt layer contributed to the decrease of deflections can better be observed in Figures 8.13 and 8.14. These figures show that the maximum deflection (in the load centre) as measured using 50 kN FWD load was linearly dependent on the temperature. At an asphalt temperature of 10°C the reduction of the maximum deflection due to the overlay equals 450 μm to 500 μm, a reduction of more than 50%.

Figure 8.13
Maximum deflections measured at different temperatures above the strain transducer 838 on the Mattingeweg

Figure 8.14
Maximum deflections measured at different temperatures above the strain transducer 837 on the Mattingeweg

8.4.2 Back-calculation analysis

The analyzed pavement structure is modelled by a linear-elastic model as it is illustrated in Figure 8.15. By means of this model the flexible pavement structure is simplified by the application of material models according to the classical linear-elastic multi-layer system, shown in Figure 8.1. In other words, the materials have been considered as linear-elastic, homogeneous and isotropic. The maximum number of model layers is equal to the number of geophones minus one. Consequently, one 1.0 m thick EPS layer was introduced instead of two layers of 0.5 m EPS30 and 0.5 m EPS25 respectively.

The deflection bowls have been back-calculated using the computer program BISAR. The differences between the measured deflections, corrected to a 50 kN load, and the deflections computed by BISAR were always within 3%. If the difference is larger the back-calculated E-values are less representative.
8.4.3 Back-calculated layer E-moduli

The back-calculated E-values of the layers in the pavement structure of the Matlingeweg on the measurement points 838 and 837 are listed in Tables 8.2 and 8.3 respectively.

<table>
<thead>
<tr>
<th>Asphalt temp.</th>
<th>Matlingeweg - point 838</th>
<th>asphalt</th>
<th>base</th>
<th>sand</th>
<th>EPS</th>
<th>subsoil</th>
<th>DIFF [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>[°C]</td>
<td>back-calculated E modulus [MPa]</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>Oct'90</td>
<td>9.0</td>
<td>17500</td>
<td>85</td>
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<td>10.4</td>
<td>60</td>
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<tr>
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<td>3.5</td>
<td>20000</td>
<td>80</td>
<td>50</td>
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<td>67</td>
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<td>25000</td>
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<td>150</td>
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<td>69</td>
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<td>24.4</td>
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<td>70</td>
<td>13</td>
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<td>2.8</td>
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<td>80</td>
<td>16.5</td>
<td>66</td>
<td>1.2</td>
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</tbody>
</table>

* - before overlaying
DIFF - maximum difference between a back-calculated and a measured deflection

Table 8.2 - Back-calculated E moduli of the layer materials in the flexible pavement structure of the Matlingeweg, based on FWD results on the measurement point 838
Measurements on a Flexible Pavement Structure

<table>
<thead>
<tr>
<th>Asphalt temp. [°C]</th>
<th>Matlingeweg - point 837</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>asphalt</td>
</tr>
<tr>
<td>Oct'90*</td>
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</tr>
<tr>
<td>Dec'90*</td>
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<td>Jan'91</td>
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<tr>
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<td>9.0</td>
</tr>
<tr>
<td>Aug'92</td>
<td>24.5</td>
</tr>
<tr>
<td>Jun'93</td>
<td>27.8</td>
</tr>
</tbody>
</table>

* - before overlaying
DIFF - maximum difference between a back-calculated and a measured deflection

Table 8.3 - Back-calculated E moduli of the layer materials in the flexible pavement structure of the Matlingeweg, based on FWD results on the measurement point 837

The back-calculation analysis was carried out using the FWD results obtained on two points, the point 838 and the point 837, as input data. These points were the first (838) and the last measurement point (837) during each of the 10 performed FWD surveys. In this way the measurements were chosen for which the largest temperature difference could be expected. The largest difference between the asphalt temperature at the beginning and at the end of the FWD survey amounted to about 6.5°C (during the summer day in August 1991). The first two measurements were performed on the pavement structure before overlaying (total asphalt thickness 130 mm) while the remaining results are valid for the overlaid pavement structure on the Matlingeweg (total asphalt thickness 210 mm).

8.4.4 Concluding remarks on FWD analysis results

Two different cases could be distinguished, before and after overlaying. The back-calculated E-values for the roadbase layers under the asphalt layer of 130 mm (before overlaying) were critically low. These values were just a fraction of the E-value usually assumed for these materials in the scope of pavement design. Such a low effective stiffness of the granular materials points out the very restricted support from the roadbase to the asphalt layer. As indicated in Chapter 6 the granular materials have a stress-dependent behaviour. The stiffening of these materials is achieved if confining stresses develop in the roadbase.
Measurements on a Flexible Pavement Structure

Low stiffness of the EPS in the sub-base means a handicap for above laid sand and crushed masonry/concrete to develop confining stress under loading. The inability of the EPS to provide a proper support to the roadbase material and the resulting low effective stiffness of the sand and the crushed masonry/concrete in combination with a relatively thin asphalt layer contributed to a large extent to the rapid pavement collapse under the heavy traffic loading. As has been indicated in Chapter 6 this could have been true seen if the design analysis was based on non-linear analyses using the mesh that has been presented in Chapter 6.

The second case was the pavement structure after overlaying with the 210 mm thick asphalt layer. The back-calculated E-values for the roadbase layers in this case had values from 140 to 600 MPa (crushed masonry/concrete base) and from 70 to 150 MPa (sand). The values found in the roadbase after overlaying are still much lower than could be expected from the considered unbound material [5]. In the case of sand, the resulting modulus values are better in line with those which can be expected for this material.

In both cases the back-calculated E-value of the EPS sub-base ranges between 10 and 20 MPa which is somewhat higher than the elasticity modulus found in compression tests for the considered EPS types EPS25 and EPS30.

The back-calculated E-value of the asphaltic concrete top layer was obviously strongly dependent on the asphalt temperature during the FWD measurements. The back-calculated E-value of the asphalt top layer varied between 25,000 (asphalt temperature 0.3°C to 0.6°C) and 5,000 MPa (T_{asph} = 31.0°C).

The E-values in the roadbase materials follow the trend of the modulus calculated for the asphalt layer. The moduli of elasticity of the roadbase layers decrease in periods with higher temperatures in accordance with a simultaneous decrease of E_{asph}-values. An opposite trend was expected because of the stress-dependent behaviour of unbound materials in the roadbase ([6] and [7]). In the cases when the E-value of the asphalt decreases (at higher temperatures) the stresses in the roadbase increase. Consequently, its (resilient) modulus should increase because of stress dependency. However, just the opposite trend can be noticed from Tables 8.2 and 8.3. The explanation for this seems to be that EPS did not provide sufficient support to the unbound material in the case of larger stress values. Therefore no high all-around confining stresses, which lead to stiffening of the unbound roadbase materials, could develop in the unbound materials.

8.5. ASPHALT STRAINS

8.5.1 Asphalt strain measurement results

The tensile strains measured at the bottom of the asphalt layer due to the FWD loading are dependent on the asphalt temperature. Figures 8.16 to 8.17 show the tensile strains measured (and corrected to the 50 kN FWD load) by means of the transducers 838 and 837 built-in in the Matlingeweg.
Measurements on a Flexible Pavement Structure

**Figure 8.16**
Horizontal tensile strains measured by means of the transducer 838 at the bottom of the asphalt layer of the Matlingeweg

**Figure 8.17**
Horizontal tensile strains measured by means of the transducer 837 at the bottom of the asphalt layer of the Matlingeweg

The minimum asphalt temperature during the strain measurements was equal to 0.3°C (January 1991) while the maximum temperature was 31.0°C (August 1991). Figures 8.18 and 8.19 show the same strain values as in Figures 8.16 and 8.17 but this time the strains are plotted against the corresponding asphalt temperatures.

**Figure 8.18**
Horizontal tensile strains measured by means of the transducer 838 as a function of the asphalt temperature

**Figure 8.19**
Horizontal tensile strains measured by means of the transducer 837 as a function of the asphalt temperature
It is obvious that this relation shows more scatter than the deflection vs temperature plot but a strong dependence of the asphalt strain on temperature can be observed. From this figure it can also be observed that the overlay produced a reduction in strain level of 80 to 100 μm/m at a temperature of 10°C. If this relation is used to estimate the tensile strain at an asphalt temperature of 20°C (normally this temperature is used for design purposes) then a measured asphalt strain of about 210 μm/m is predicted for the pavement structure before overlaying and about 80 μm/m after overlaying. This high asphalt strain estimated for the original structure would result into a rather short pavement life expressed as the number of 50 kN load repetitions. The asphalt strain reduction due to the overlay can be roughly estimated at about 110 to 150 μm/m at the reference asphalt temperature of 20°C.

8.5.2 Asphalt strains at the reference temperature

The previously described analysis allowed a rough comparison of the strains at different asphalt temperatures, although the development of the asphalt strains in time due to the traffic loading could not be determined in this way. Transformation of the measured strain values to the corresponding values at the reference temperature of 20°C was carried out by means of an established relationship between the back-calculated E-values of the asphalt layer (Tables 8.2 and 8.3) and the corresponding asphalt temperatures (Figure 8.20).

One should keep in mind the way in which the asphalt temperature was determined. No built-in thermocouples were available so that the temperature was measured in a narrow borehole with a depth of approx. 50 mm. The way the measured temperatures have been corrected is described in subsection 8.2. These corrected temperature values, shown in Figures 8.16 and 8.17, have been considered as relevant for the asphaltic concrete layer of the Mattingweg. In this case the $S_{\text{mix}} = 10,500$ MPa for $T_{\text{ref}} = 20^\circ\text{C}$.

As an illustration the chart of a standard Dutch asphalt mix is also shown in Figure 8.20. It is the gravel asphaltic concrete used in road pavement structures exposed to traffic class 3 and 4 [8].

![Fig. 8.20 - Relation between asphalt mix stiffness and asphalt mix temperature](image.png)
The next step consists of BISAR calculations of the tensile strains at the bottom of the asphalt layer using the back-calculated E-values of Tables 4.1 and 4.2. The resulting strain values are marked as $\epsilon_{\text{calT}}$ (cal = calculated, T = temperature). Correlation between the calculated asphalt strains $\epsilon_{\text{calT}}$ and the corresponding measured strains $\epsilon_{\text{mT}}$ is shown in Figure 8.21.

**Figure 8.21**

Correlation between the measured strains at the bottom of the asphalt layer and the corresponding strain values calculated by means of the BISAR program

BISAR calculations were performed again with the value $S_{\text{mix}} = 10,500$ MPa as the input E-value for the asphalt layer. The corresponding E-values from Tables 8.2 and 8.3 were used for the remaining pavement layers. Consequently, the stress-dependent behaviour of base materials was neglected but no alternative method existed. Resulting calculated strains had the mark $\epsilon_{\text{cal2}}$. In Figure 8.22 the calculated asphalt strain for the asphalt temperature at 20°C is shown as a function of time. Before overlay, the $\epsilon_{\text{cal2}}$ asphalt strains were about 192 $\mu$m/m, a very high value. This high asphalt strain resulted in a short pavement life of the original structure. After overlay, the asphalt strain was equal to about 85 $\mu$m/m. Furthermore, no increase in the asphalt strain in time after the construction of the overlay points to a good condition of the pavement structure.

From Figure 8.22 it can be observed that the overlay produced a reduction in strain level of approximately 105 $\mu$m/m, which is somewhat lower than the values estimated on the basis of Figures 8.18 and 8.19 in the first section of this chapter. Nevertheless, the design pavement life of the overlaid structure expressed in the allowable number of standard axle load repetitions is many times longer than the life of the original pavement structure.
8.5.3 Concluding remarks on asphalt strains

The strain level at the bottom of the asphalt layer was equal to approximately 192 μm/m (at the reference temperature of 20°C) before the overlay. The design life of the original pavement structure, roughly estimated on the basis of the fatigue curves for the gravel asphaltic concrete in [8], amounts to only a few thousands of 100 kN standard axle-load repetitions. Notwithstanding the inability to determine the pavement design life in a more accurate way (from fatigue testing of the asphaltic concrete actually applied) it was obvious that the pavement structure on the Matlingeweg had a very short design life before the overlay. The main reason for the improper pavement design was the use of overestimated E-values for the unbound materials in the roadbase.

The asphalt strain at the reference temperature (20°C) remained more or less constant in the 3 year period after the overlay. The constant value of the strain is a sign of a good condition of the pavement structure although it should be mentioned that the time period of observation is short. The horizontal tensile asphalt strain amounts to about 85 μm/m at the reference temperature of 20°C.

Back-calculated E-values for the roadbase material of the Matlingeweg point to a decrease with the temperature of the effective stiffness of this material built-in above the EPS because of the lack of support from the sub-base. In order to design an appropriate roadbase thickness the minimum E-values from literature should be used as input data in calculations of the design life of pavement structures with an EPS sub-base.
8.6 CONCLUSIONS ON MEASURING RESULTS

- The back-calculated very low E-values for the sand (from 40 to 65 MPa) and the crushed masonry/concrete (from 80 to 85 MPa) before the overlay highlight the inability of the EPS to provide a proper support to the roadbase in the considered pavement structure with a 130 mm thick asphalt layer. Correspondingly insufficient support of the roadbase to the asphalt layer resulted in a critically high asphalt strain of about 192 μm/m (T = 20°C). Use of overestimated E-values for the roadbase materials was probably the main reason for the inappropriate pavement design.

- Open joints between the EPS blocks in a sub-base can have very serious consequences for the design life of pavement structures, and those have to be avoided by all means. The joints between the blocks in various layers should not coincide with each other. The open joints are especially risky in the case of an EPS sub-base which consists of only one EPS layer. The longitudinal joints between the EPS blocks should not be close to a wheel track. An adequate (lateral) support of the blocks is necessary to prevent any movement of the blocks.

- The back-calculated E-value of the EPS sub-base ranges from 10.4 to 19.7 MPa, which is somewhat higher than the elasticity moduli found in literature for the EPS types under consideration (EPS25 and EPS30).

- The back-calculated E-value of the asphaltic concrete layer varied between 25,000 and 5,000 MPa, due to the temperature range of 0.2°C to 31.0°C, during the various FWD measurements.

- The E-value of the crushed masonry/crushed concrete roadbase ranged from 140 to 600 MPa after the overlay. The modulus found for the sand varies between 70 to 150 MPa. In some measurements the values found for the crushed masonry/concrete were much lower than could be expected for this unbound material. In order to design an appropriate roadbase thickness the minimum E-values from the literature should be used for the considered unbound material as input data in calculations of the design life of the pavement structures with an EPS sub-base.

- The results obtained clearly indicate the importance of the development of lateral compressive stresses in unbound materials in order to obtain sufficient stiffness. As was indicated in Chapter 6 these lateral stresses will not develop in pavement structure types as used in the Matlingeweg. It was also indicated in Chapter 6 that a high modulus capping layer on top of the EPS would significantly improve the performance of pavements with an EPS sub-base.

- The asphalt strain remained more or less constant in the 3-year period after the overlay was placed. The constant value of the strain is a sign of a good condition of the pavement structure. The maximum horizontal tensile asphalt strain amounts to about 85 μm/m at the reference asphalt temperature of 20°C.
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9.1 INTRODUCTION

This chapter discusses the three-dimensional (3-D) finite element analysis of pavement structures with an EPS sub-base layer. Three dimensional modelling is important since it allows modelling of the block structure in the EPS sub-base, in contrast with two-dimensional or axial symmetric pavement models. Three separate 3-D analyses were carried out. Firstly, a 3-D pavement structure model was developed with a single vertical interface layer next to the wheel load. Secondly, a polder road was analyzed; in this case a much more complex model was developed to analyze the effects of: a) different block patterns, b) various EPS types in the sub-base and c) a concrete capping layer, on the stress and strain values in the pavement layers. Finally, using experiences from the previous analyses, a model for a motorway pavement structure was designed with a simplified EPS block structure to investigate the consequences of implementation of EPS (instead of sand) on performances of (Dutch) motorway pavement structures.

The analysis that is described in section 9.2 was performed to investigate whether the existence of an open joint in the sub-base does affect the pavement behaviour. In this particular analysis one interface layer was used to model the vertical joint. The wheel load was placed just adjacent to the joint to enforce maximum shear forces in the layer above the EPS sub-base. Also the effects of using a concrete (capping) layer above the EPS sub-base and a somewhat different EPS type were determined.

Section 9.3 deals with the analysis of a polder road. In the western part of the Netherlands a great number of polder roads are located in areas with a low bearing capacity subsoil. These polder roads, constructed in the traditional way on soft, saturated subsoil, are subjected to (uneven) settlements. The use of EPS, particularly in the sub-base of these roads, is likely to offer a solution for the settlement problems by reducing the weight of the pavement structure. Once designed, the 3-D polder road model enabled the analysis of the effects of different block patterns in the sub-base on the pavement structure behaviour. The complexity of the model was defined by the need of modelling different block patterns by means of a single mesh.

The last finite element analysis, the subject of section 9.4, was performed on a model for a motorway pavement structure with layer thicknesses corresponding to the usual values for Dutch motorways. 3-D modelling of the heaviest loaded road type was done to determine to which extent building-in of EPS blocks in the sub-base influences its structural behaviour. The reference was an identical structure with a sand sub-base layer. Additionally, the effects of a concrete capping layer above the EPS blocks were investigated. Based upon the results of the previous analyses the motorway model was simplified compared to the polder road model. A single vertical joint was designed in the EPS layer and the axle load was placed next to that joint.

The most characteristic stress and strain values as well as the conclusions are presented at the end of the chapter.
All described models and analyses in Chapter 9 were realized by means of the three-dimensional version of the finite element program CAPA (Computer Aided Pavement Analysis) [1]. The skills implemented in the program enabled a sophisticated pavement analysis in a relatively user-friendly way. Amongst others, the included mesh generator facilitated mesh forming and, in postprocessing, the optionally selected results could be presented in graphs. The implemented interface elements allowed a flexible simulation of mechanisms in joint faces.

9.2 SINGLE-INTERFACE PAVEMENT STRUCTURE MODEL

In situ measurements (Chapter 7 and 8) and numerical analyses with linear-elastic multilayer systems [2] have contributed to a better insight into the structural response of pavement structures with an EPS sub-base. Amongst others in-time development of surface deflections was measured (on various types of pavement structures) as well as the development of tensile strains at the bottom of the asphalt layer (in one flexible pavement). In this way the effective stiffness of the individual pavement layers and the overall condition of the pavement structures after several years of traffic were studied. The measurements indicated some locations of weakness in the pavement structures with high surface deflections and rutting. In the case of the pavement structure in Rotterdam with a thin asphalt toplayer, described in section 8.3, longitudinal cracking occurred.

The above-mentioned longitudinal cracking in the flexible pavement structure occurred within a month after the structure was completed (subsection 8.3.5). It illustrates the potential danger associated with an inadequate design and/or construction of pavements with an EPS sub-base. It appears from in-situ observations that pavement failure could be caused by wide gaps in the joints of adjacent EPS blocks (Figure 8.8). The location of these longitudinal joints in the cross section coincided with the position of the wheel track. Simulation of these events and in particular of the influence of the presence of joints between the EPS blocks on the load transfer mechanisms of the pavement structure was the reason for the pavement analysis that will be presented in this section.

Simulation of open joints and cracks makes it necessary to implement appropriate elements which can simulate discontinuities in the pavement layers. For this reason the three-dimensional finite element program CAPA-3D [1] was utilized. In CAPA-3D simulation of joints between EPS blocks and/or cracks can be achieved by means of interface finite elements.

In the following subsections the geometry, the material properties and the load characteristics of the simulated pavement structures are reviewed. Three different stages of crack development in the body of the pavement have been simulated by means of the CAPA-3D program. The deformations, stresses and strains at critical pavement locations for each of these cases are presented and, on the basis of these, conclusions are drawn and recommendations for design are made.
9.2.1 3-D Finite element mesh

The initial and final CAPA-3D finite element meshes of the pavement model are shown in Figure 9.1. The final mesh is generated from the initial mesh that consists of the so-called 'superelements'. Within these superelements only one material type can be implemented. The element division, fine in the loadspreading area, becomes coarser with increasing distance from the load centre.

The boundary conditions are defined in the outer nodes of the superelements where the displacements in horizontal and (only at the bottom) vertical directions were restrained. The dimensions of the mesh are 14.7 m by 5.55 m (3.0 m at the top). Such a large vertical dimension of approximately 15 m is chosen to prevent any influence of the rigid bottom on the loadspreading inside the pavement model. The height of the mesh is approximately equal to the dimensions recommended in the literature [3] for finite element pavement models. For the same reason the horizontal dimension of the mesh is taken as 5.55 m. The dimension in the longitudinal direction is 1.8 m which coincides with the distance between the load centre and the last geophone belonging to the standard configuration of a falling weight deflectometer. In this way it was possible to draw complete deflection bowls at the pavement surface in the cross and longitudinal direction of the road.

9.2.1.1 Analyzed pavement structure

The layer thicknesses in the pavement model have been chosen such that they are close to those of the Matlingeweg in Rotterdam (subsection 8.3.4) where severe cracking developed within a month after the first stage of reconstruction. The lay-out of the modelled pavement structure is shown in Figure 9.2. The top-layer of asphaltic concrete, the unbound base layer and the sand layer have a thickness of 150, 400 and 150 mm respectively while the sub-base consists of a 1.0 m thick EPS layer.
9.2.1.2 Materials parameters

The $E$ moduli and Poisson's ratios of the layers are assumed to be similar to the back-calculated values described in subsection 8.3.4. Only for the peat a somewhat lower value of 30 MPa is assumed. The $E$-modulus of EPS has a value of 5 MPa while the $E$-values for the sand, base material and asphalt layer are equal to 100, 300 and 10,500 MPa respectively. The Poisson's ratio is 0.35 for all materials except for EPS where $\nu_{EPS}=0.1$; this value was found in the materials research program (subsection 5.6.2).

9.2.1.3 Modelled loading and vertical interface elements

The load is a 50 kN load through a circular area with a diameter of 300 mm. Because of symmetry only half of the described loading is modelled (Figure 9.4). The contact stress of 0.707 MPa is applied next to the vertical interface layer.

The position of the vertical interface layer next to the loading area represents the most critical configuration which can appear in situ. The vertical interface layer represents a joint between the EPS blocks; the load is located exactly next to the longitudinal open joint. Because of the open joint in the EPS sub-base the unbound base material is not properly supported. This leads to an improper support of the asphalt toplayer by the roadbase. On the Matlingeweg longitudinal cracking in the asphalt was observed because of this.
The implemented interface elements not only allow an open joint between the EPS blocks to be simulated but also a crack in the asphalt layer and a no-tension response of the unbound materials in the base layers.

9.2.2 Resulting pavement structure deformations

In the following the deformations for three essentially different situations are presented. The first case describes the situation with no crack in the asphalt toplayer and a monolithic EPS sub-base. In the second case an open joint is modelled in the EPS layer. At the same time the unbound base and the sand layer above the EPS sub-base are modelled as no-tension material. In the third case the pavement structure is analyzed with an open joint in the sub-base, a crack in the asphalt layer and the no-tension response of the unbound materials.

The calculated deformations in the first case without 'active' interface elements are presented in Figure 9.5. Only very small deformations due to a 50 kN loading occur in the pavement structure.
The deformations in the pavement structure with an open joint between the EPS blocks and the no-tension response of the unbound materials in the layers above the EPS sub-base are shown in Figure 9.6. The asphalt layer was modelled to be continuous without a crack next to the loading area.

Fig. 9.6 - Deformed mesh due to the 50 kN load of the pavement model where an open joint between EPS blocks but not a crack in the asphalt layer are modelled

The effects of the open joint are reflected in the widening of the interface elements in the unbound roadbase as a result of tensile stresses. The most significant result is additional widening of the joint between the EPS blocks. The support of the EPS sub-base to the above laid roadbase is reduced due to the further opening of the existing joints between the EPS blocks. The insufficient support to the unbound base is reflected further in the insufficient support of the roadbase to the asphalt layer. Consequently, higher stress and strain values occur in the asphalt layer under the load.

The deformations in the pavement structure with both an open joint between the EPS blocks and a crack in the asphalt layer are graphically presented in Figure 9.7.

Fig. 9.7 - Deformed mesh due to the 50 kN load of the pavement structure with both an open joint between the EPS blocks and a crack in the asphalt top layer
The crack in the asphalt layer leads to large differences in deformations between the 'loaded' and 'unloaded' part of the pavement structure. The occurring vertical deformations in the EPS block below the load are relatively bigger than the values on the other side of the joint.

### 9.2.3 Calculated stress and strain values

In this paragraph the horizontal stress ($\sigma_{x}^{ph}$) and strain values ($\varepsilon_{x}^{ph}$) at the bottom of the asphalt layer and the vertical stress values at the top of the EPS sub-base ($\sigma_{zz}^{EPS}$) are listed for the cases described in the previous section (1). Additionally, the analyses were repeated for pavement structure models with, on the one hand, a concrete capping layer instead of a sand layer above the EPS sub-base (2) and, on the other hand, with a somewhat heavier EPS type in the sub-base (3).

1. **Unbound base and sand layer above EPS sub-base**

<table>
<thead>
<tr>
<th>$\varepsilon_{x}^{ph}$ [(\mu m/m)]</th>
<th>$\sigma_{x}^{ph}$ [MPa]</th>
<th>$\sigma_{zz}^{EPS}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>neither an open joint between EPS blocks nor a crack in the asphalt layer</td>
<td>108</td>
<td>1.76</td>
</tr>
<tr>
<td>an open joint between EPS blocks but no crack in the asphalt layer</td>
<td>138</td>
<td>2.14</td>
</tr>
<tr>
<td>both an open joint between EPS blocks and a crack in the asphalt layer</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

2. **Cement treated capping layer above EPS sub-base**

(thickness 150 mm, $E=11,000$ MPa, $\nu=0.35$)

<table>
<thead>
<tr>
<th>$\varepsilon_{x}^{ph}$ [(\mu m/m)]</th>
<th>$\sigma_{x}^{ph}$ [MPa]</th>
<th>$\sigma_{zz}^{EPS}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>neither an open joint between EPS blocks nor a crack in the asphalt layer</td>
<td>89</td>
<td>1.45</td>
</tr>
<tr>
<td>an open joint between EPS blocks but no crack in the asphalt layer</td>
<td>90</td>
<td>1.47</td>
</tr>
<tr>
<td>both an open joint between EPS blocks and a crack in the asphalt layer</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

3. **Unbound base layers above heavier type EPS**

($E_{EPS}=10$ MPa instead of $E_{EPS}=5$ MPa)

<table>
<thead>
<tr>
<th>$\varepsilon_{x}^{ph}$ [(\mu m/m)]</th>
<th>$\sigma_{x}^{ph}$ [MPa]</th>
<th>$\sigma_{zz}^{EPS}$ [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>neither an open joint between EPS blocks nor a crack in the asphalt layer</td>
<td>106</td>
<td>1.73</td>
</tr>
<tr>
<td>an open joint between EPS blocks but no crack in the asphalt layer</td>
<td>128</td>
<td>2.02</td>
</tr>
<tr>
<td>both an open joint between EPS blocks and a crack in the asphalt layer</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
9.2.4 Concluding remarks on single-interface analysis

The presence of open joints between the EPS blocks in the sub-base significantly affects the behaviour of pavement structures with an unbound roadbase. The insufficient support to the unbound base reflects in the insufficient support of the roadbase to the asphalt layer. Consequently, higher stress and strain values occur in the asphalt layer which result in a shorter design life of the pavement structure. If the strain values become critical, this leads to cracking of the asphalt. After cracking of the asphalt layer, a wheel load on one side of the crack will result in a relatively larger vertical deformation in the EPS block below the load than in the blocks on the other side of the joint.

Implementation of a concrete capping layer above the EPS sub-base neutralizes the negative influence of the block joints on the pavement behaviour. Such a capping layer ensures enough support to the unbound roadbase layer above this layer, also in the case of existing open joints between the EPS blocks. Construction of a concrete capping layer contributes to a significant increase of the pavement design life.

Application of a heavier EPS type with a somewhat higher elasticity modulus in the sub-base has only a very limited influence on the horizontal strain values at the bottom of the asphalt layer and, therefore, on the overall behaviour of the pavement structures with an EPS sub-base. The use of a heavier EPS type is therefore no solution in solving the effects of potential construction errors like open joints.

9.3 POLDER ROAD ANALYSIS

Polder roads in the western part of the Netherlands, constructed often on a thick peat layer, are likely to be subjected to (uneven) settlements demanding frequent maintenance. The great number of polder roads on subsoils with poor characteristics, on one hand, and the use of EPS in the sub-base as a solution of the settlement problems, on the other, were two reasons for performing a 3-D analysis on these pavements.

9.3.1 Polder road cross section

Polder roads belong to the category of narrow rural roads. Dutch standards prescribe the cross section for these lowest road categories. They are given in Figure 9.8. The pavement width is 4.5 m and 3.5 m respectively, with a verge of 2.0 and 1.5 m respectively at each side of the pavement [4]. The modelled polder road has a width of 4.0 m with 1.5 m wide verges on both sides.
9.3.2 Pavement layer materials

The modelled pavement structures consist of a 150 mm thick asphalt toplayer, a 400 mm thick unbound base layer and a 150 mm sand layer placed on top of a 1.0 m thick EPS sub-base. Two other cases were analyzed as well which are, firstly, a pavement with a 200 mm thick asphalt layer and, secondly, a pavement where the sand layer was replaced by a concrete capping layer. To enable modelling of different EPS types within the sub-base the EPS sub-base is subdivided into two sublayers, each 0.5 m thick. The lay-out of the cross section of the modelled pavement structure is shown in Figure 9.9.

The elasticity moduli of the pavement layers have the following values: \( E_{\text{asph}} = 7500 \text{ MPa} \), \( E_{\text{base}} = 200 \text{ MPa} \), \( E_{\text{sand}} = 50 \text{ MPa} \), \( E_{\text{EPS}} = 5 \text{ MPa} \), \( E_{\text{peat}} = 30 \text{ MPa} \) and \( E_{\text{verge}} = 50 \text{ MPa} \). The E values for the unbound base are assumed to be relatively low. This is because the back-calculated values for the roadbase of the flexible pavement structure in the Mattingueweg (Tables 8.2 and 8.3) and E-values found in BAS test sections (Table 6.2) for unbound materials laid directly above the EPS sub-base show low values. Poisson’s coefficients were taken as 0.35 for all materials with exception of EPS with \( \nu_{\text{EPS}} = 0.1 \).

For the concrete capping layer, placed on top of the EPS layer instead of sand, an elasticity modulus of 11,000 MPa is used; in that particular case \( E_{\text{base}} = 400 \text{ MPa} \). In subsection 6.3.3 it has been shown that a cement treated layer influences the stress condition in the unbound base to such an extent that higher moduli were obtained. Furthermore, a case with \( E_{\text{EPS}} = 10 \text{ MPa} \) was analyzed to investigate the effects of a heavier EPS type.
9.3.3 3-D mesh forming

The design of the mesh was determined by the requirements to model different EPS block patterns by means of a single mesh, on one hand, and to use a minimum number of elements, on the other. The vertical interface elements (IF) are implemented only when necessary for creating the various block patterns. The horizontal interface layers are modelled in the mesh only on top of the EPS sub-base and between the EPS sub-layers. Furthermore, the interface elements are not implemented at distances larger than 2.0 m from the loading area in the longitudinal direction. It is assumed that such interface elements have no significant effects on the pavement behaviour. The cross section of the super mesh of the pavement model used is shown in Figure 9.10.

![Figure 9.10](image)

*Figure 9.10* - Cross section of the pavement structure model of the polder road with the implemented interface elements, enabling the modelling of different patterns of the EPS blocks with dimensions 0.5x1x2 m in the sub-base

The initial (super) mesh is composed of 1,684 superelements and 10,978 nodes, while the final mesh, created by division of the superelements, has 3,960 elements and 17,676 nodes. The dimensions of the mesh are 14.7 m by 12.1 m (7.0 m at the top) by 4.0 m. A large vertical dimension of approximately 15 m is chosen, identically to the mesh in the foregoing section, to minimize the influence of the rigid bottom on the load-spreading inside the pavement model. The horizontal dimension at the top of the mesh is taken as 7.0 m reflecting the cross section dimensions described in the previous subsection. In longitudinal direction the dimension is 4.0 m. According to the literature [5] the edges will not have a significant influence at this distance from the loading centre. The upper part of both the superelement mesh and the final mesh is shown in more detail in Figure 9.11.
Boundary conditions valid for the 3D finite element pavement model concern the freedom of displacements at the boundaries of the model. The nodes on the vertical outer model edges are supported in the horizontal direction. Horizontal displacements and shear stresses in the vertical direction are assumed to be zero. Consequently, only vertical displacements of these nodes are possible under the loading. The nodes at the bottom of the model are supported both in the vertical and in the horizontal direction which means that all displacements are zero. In other words, the CAPA-3D finite element model represents a road structure with a rigid bottom. The boundary conditions of the applied pavement model differ to some extent from reality. Generally speaking there is no rigid bottom at certain depth and no horizontal support at certain distance from the loading areas. The effects of the implemented boundary conditions on the stresses and strains in the model were however minimized by choosing large enough vertical and longitudinal dimensions.

### 9.3.4 Modelled axle load

The placement of the load is shown in Figure 9.12. Each load is a 50 kN load so a 100 kN axle load is simulated. For reasons of simplicity the loaded areas are assumed to be rectangular. The axle load is placed in such a way that the left wheel is immediately adjacent to a vertical joint in the EPS layers.
9.3.5 Effects of different EPS block patterns

Until now possible effects of EPS block patterns on overall performance have never been calculated by means of a 3-D finite element analysis. Based on empirical observations and engineering judgement it was assumed that there could be certain (negative) effects, especially if the joints are open. Therefore it is prescribed in the Dutch Regulations that the joints between the EPS blocks should not be wider than 10 mm [6]. Nevertheless the extent to which various EPS block patterns contribute positively or negatively to pavement structural behaviour, is subject for discussion in the next paragraphs.

9.3.5.1 Modelled EPS block patterns

The investigation of the effects of the EPS block patterns in the sub-base on the structural behaviour was concentrated on three modelled cases 1a, 2a and 3a, which are illustrated in Figure 9.13. The first and the third case represent two extreme situations, namely an unrealistic monolithic EPS sub-base without joints (case 1a) and the EPS sub-base consisting of small blocks (1×0.5×0.5 m), laid in two layers in such a way that the vertical joints overlap each other (case 3a). In the second model, case 2a, a single layer of EPS blocks with dimensions 1×1×0.5 m was modelled. In Figure 9.13 both the cross section and the longitudinal section of the pavement structures are shown.

Furthermore, the interface elements in the unbound layers above the open joints were modelled so that they can not take tensile stresses. Such no-tension response was assumed to represent real local in-situ conditions in unbound base layers which are improperly supported because of the existing open joints in the sub-base below, which results into tensile stresses in the unbound base. These stresses cannot be taken by unbound material and therefore a no-tension response was adopted.
9.3.5.2 Resulting deformations for different EPS block patterns

The deformed meshes belonging to the cases 1a, 2a and 3a are shown in Figures 9.14 and 9.15. The applied large magnification factor (mf = 500) makes the deformations look much bigger than the values really are. In this way differences in deformations are shown clearly. However it should be realized that the calculated differences are relatively small.
Figure 9.14 - Upper pavement model parts with different implemented block patterns in the sub-base, deformed due to a simulated standard 100 kN axle loading (magnification factor mf=500)

Figure 9.15 - Cross and longitudinal sections of the deformed upper parts (magnification factor mf=500) of the pavement models for the cases 1a, 2a and 3a with different block patterns in the sub-base.
9.3.5.3 Calculated deflection bowls

Deflection values due to the left-hand as well as the right-hand wheel load (the right-hand one is closest to the pavement edge) were determined (Figure 9.16). From this figure it appears that the influence of the vertical joints in the EPS sub-base is more significant than the additional division of the EPS sub-base into two sublayers.

![Figure 9.16](image-url)  
*Figure 9.16 - Calculated deflections in longitudinal direction at the positions of both left-hand and right hand loading areas of the polder road models with different EPS block patterns in the sub-base*

9.3.5.4 Resulting strain and stress values

In Figure 9.17 the values of the horizontal strain in x-direction ($\epsilon_{xx}$) at the bottom of the asphalt layer are given for cases 1a, 2a and 3a.

![Figure 9.17](image-url)  
*Figure 9.17 - Horizontal strain values, $\epsilon_{xx}$, at the bottom of the asphalt layers of the pavement models with different EPS block patterns in the sub-base (case 1a, 2a and 3a)*
There is a distinct difference between the strain values calculated in case 1a with a monolithic EPS sub-base and the strain values of the pavement structures where the joints between EPS blocks (case 2a and 3a) were modelled. Existence of vertical joints in the EPS sub-base combined with a critical position of a wheel load above such a joint seems to lead to about 10% higher horizontal tensile strain values under the 100 kN standard axle load. An approximately 10% higher tensile strain in the asphaltic concrete could mean a significant reduction of the pavement design life (up to 40%) if longitudinal open joints coincide with a wheel track.

Furthermore it appears from Figure 9.17 that the influence of the additional division of the EPS layer into two sublayers (case 3a) on the asphalt strain values is very much less than the effects of the vertical joints (case 2a). Actually, no influence was noticed when the horizontal interface elements in the middle of the EPS sub-base were activated (case 3a).

The question is whether it would make a difference if vertical joints in the two subsequent EPS sublayers had not overlapped each other. To find the answer to this question a fourth block pattern was modelled, case 4a, with EPS blocks with realistic dimensions 1x2x0.5 m. Figure 9.18 shows how the block structure was designed in the EPS sub-base.

![Figure 9.18](image)

**Figure 9.18** - Cross and longitudinal sections of the pavement structure model with a realistic block pattern in the EPS sub-base, consisting of blocks with dimensions 1x2x0.5 m laid in two sublayers.

Comparison of both the deflections at the pavement's surface along the wheel tracks and horizontal strain values ($e_{xx}$) at the bottom of the asphalt layer for cases 2a and 4a showed that no significant influence of different block patterns could be observed. This can be seen in Figure 9.19.
9.3.6 Implementation of different EPS types

The pavement structures described in Chapter 7 and 8 contain various EPS types in their sub-bases. Both EPS15 and EPS20 were used in the concrete block pavement structure in Stolwijk (subsection 7.3.4) while EPS25 and EPS30 blocks were laid under the Mattingeweg in Rotterdam. Diversity of EPS types used in these pavement structures is illustrative for the current Dutch situation. As already mentioned (sub-section 8.3.3) the question is whether use of denser EPS types, which have only a few MPa higher elasticity modulus than the low density EPS types, justifies the accompanying extra costs. As a contribution to the solution of this dilemma, the calculations on the polder road model (case 3a) were repeated with an elasticity modulus for the EPS layer of 10 MPa instead of the previously used 5 MPa.

The horizontal strain values in the asphalt toplayers on EPS sub-bases of different quality are shown in Figure 9.20. The differences in strain values were that small that they could not be made visible in the graph in spite of the fact that the modulus of the EPS differed with a factor 2. Such results are similar to the findings of the previous analysis on the single-interface model (section 9.2). Therefore it may be concluded that the application of denser (and more expensive) EPS types with a somewhat higher elasticity modulus has no significant effect on the overall pavement behaviour. Application of a denser EPS is therefore hardly justifiable.
9.3.7 Concrete capping layer above EPS sub-base

In the structures analyzed so far, the asphalt strains as well as the deflections have high values, pointing out that the design life of such pavement structures would be limited. Although the traffic intensity on the polder roads is usually low, high axle loads due to heavy agricultural traffic can occur causing rapid deterioration of the pavement structure. Therefore it can be necessary to apply either a thicker asphalt layer or, instead of the sand layer, a concrete capping layer above the EPS sub-base.

The effects of the above mentioned improvements were studied for the polder road model (case 3a). Firstly, the asphalt layer thickness was increased from 150 to 200 mm (case 3b) and, secondly, maintaining the original asphalt layer thickness of 150 mm, the sand layer above the sub-base was replaced by a loadspreading concrete capping layer with E = 11,000 MPa (case 3c). Figure 9.21 shows the horizontal asphalt strains calculated for the three cases (3a, 3b and 3c).

The use of a 50 mm thicker asphalt toplayer, i.e. 200 mm instead of 150 mm, and a 150 mm thick concrete capping layer above the EPS sub-base appeared to be similarly beneficial with respect to the horizontal asphalt strain. The realized asphalt strain reduction is approximately 30%. Such a strain reduction at the bottom of the asphalt layer means a multiplication of the design life in terms of fatigue of the asphalt layer by a factor of approximately 6.
9.3.8 Concluding remarks on polder road analysis

Existence of open vertical joints in the EPS sub-base results into approximately 10% higher horizontal tensile strain values at the bottom of the asphalt layer under the 100 kN standard axle loading when a wheel load is located above such a joint. Accordingly, if longitudinal open joints coincide with a wheel track a significant reduction (about 40%) of the pavement design life might be expected.

EPS sub-bases where the blocks are laid in various patterns deform somewhat different under traffic loads. Different sub-base behaviour occurs along the joints between the EPS blocks. However, the absolute deformation differences are so small that the effects on the structural pavement behaviour, i.e. the asphalt strain, is of no practical importance.

The calculations showed that the influence of dividing the sub-base into two sublayers seems to be very limited.

The application of denser (and more expensive) EPS types with a somewhat higher elasticity modulus seems not to result into a significantly better overall pavement behaviour. For the analyzed example, strengthening of the polder road pavement by increasing the asphalt layer thickness with 1/3 (extra 50 mm) or by building-in a 150 mm thick concrete capping layer above the EPS sub-base appeared to be similarly beneficial with respect to the horizontal asphalt strain. The realized asphalt strain reduction amounts to approximately 30% resulting in an extension of the asphalt pavement design life by a factor of approximately 6.
9.4 MOTORWAY ANALYSIS

Finally a 3-D finite element analysis was performed on a motorway pavement. Being the most heavily loaded road category, motorways demand implementation of pavement structures with outstanding bearing capacities. In the western part of the Netherlands many parts of the motorways have been constructed on poor subsoils. Especially in case of widening, this condition is somewhat troublesome because widening of the existing embankments results in settlements and possibly even in stability problems. More generally it is of interest to investigate the possibilities of designing a proper motorway pavement structure with a sub-base of light-weight EPS blocks, the application of which would strongly reduce the stability and settlement problem.

9.4.1 Motorway cross section

According to the Dutch Guidelines for Design of Motorways [8] the standard cross section of a motorway with two times two traffic lanes is as illustrated in Figure 9.22. The traffic lanes are 3.5 m wide with a 3.0 m wide emergency lane and a 3.0 m wide verge. In the modelling of the standard motorway cross section, attention was focussed on the outer traffic lane which is used by heavy trucks.

![Figure 9.22](image)

Standard Dutch 2x2 lanes motorway cross section according to the Dutch Guidelines for Design of Motorways

9.4.2 3-D mesh and simulated axle loading

The mesh that was used for the motorway pavement analysis is shown in Figure 9.23. Based on the results obtained in the previous section, the inner traffic lane was not completely modelled (compare Figures 9.22 and 9.23).

A 100 kN standard axle load was applied by means of two rectangular load areas, in the identical way as on the polder road model (subsection 9.3.4). The outer 'wheel', i.e. the right-hand loaded rectangle, was modelled at a distance of 0.1 m from the emergency lane instead of the road edge. Similar to the previously analyzed pavement models
the load was located next to an interface element to enable the analysis of the effect of joints in the EPS layer.

Previous investigation on the complex polder road model pointed out that the effects of the open joint next to the load are dominant compared to the effects of other EPS layer parameters with respect to the maximum stress and strain values in the asphalt layer. Based on this conclusion it was possible to reduce the complexity of the motorway model without losing representativeness regarding the relevant stress and strain values. The simplification consisted of implementation of only one vertical interface layer located next to the right-hand loading area of the axle load. Figure 9.24 shows the complete motorway model with its dimensions. The height of the model almost equals 15 m and was motivated with the reasons explained in subsections 9.2.1 and 9.3.3.

9.4.3 Pavement layer materials

The modelled pavement had layer thicknesses identical to one of the standard structures applied on Dutch motorways [9]; they are given in Figure 9.25. The biggest difference in comparison with the previously investigated models was a 260 mm thick toplayer of asphaltic concrete. The unbound base (300 mm), the sand layer (100 mm) and the EPS sub-base (1 m) had thicknesses more or less similar to those modelled in foregoing cases.
Materials parameters, i.e. the elasticity moduli and Poisson's coefficients were the same as in the polder road model (subsection 9.3.2) with the exception of the unbound base material, i.e. crushed concrete, above a pure sand sub-base. Again, a reduced effective stiffness of the unbound base materials laid directly above the EPS sub-base was taken into account (200 MPa instead of 500 MPa).

9.4.4 EPS sub-base vs. sand sub-base

Traditionally, the sub-base in Dutch motorways consists of sand so that the pavement structure with exclusively a sand layer below the roadbase may be considered as the reference case. Therefore, the analysis of the structural behaviour of the motorway models started with a comparison between the case with a sand sub-base and the case with an EPS sub-base.

9.4.4.1 Resulting deformations in the upper motorway layers

The calculated deformations, magnified by the same number to become noticeable, are shown in Figures 9.26 and 9.27.

Figure 9.26 (1st part)
From Figures 9.26 and 9.27 it can be concluded that the structural behaviour changes if an EPS sub-base (with an open joint) is applied.
9.4.4.2 Calculated deflections and asphalt strain values

Deflections at the pavement’s surface and the calculated strain values at the bottom of the asphalt layer are graphically presented in Figure 9.28.

![Diagram showing deflections and strain values](image)

Figure 9.28 - Deflections at the pavement’s surface and the strain values at the bottom of the asphalt layer for the motorway models with the pure sand sub-base and with the EPS sub-base (this last either with a monolithic layer or including an open joint)

The horizontal strain values at the bottom of the asphalt layer of the motorway model with the sand sub-base are approximately 40% lower than the corresponding values in the pavements with an open joint in the EPS sub-base. As a result, the reduction of the pavement design life would be about 12 times, that is if the EPS sub-base has been built-in with an open joint exactly along the wheel track. It is obvious that such a critical configuration with open joints should not occur in reality. Still, as illustrated on the right-hand part of Figure 9.28, even building-in a monolithic EPS sub-base instead of the sand sub-base results into approximately 25% higher maximum asphalt strains which results in a reduction of the pavement life with a factor of about 3. For the motorways with extreme traffic intensities these reduced numbers of allowable axle load repetitions might not be acceptable and pavement strengthening could be necessary. A possible way to improve the performance of a motorway pavement with an EPS sub-base is the application of a concrete capping layer on top of the EPS sub-base. The effects of such a layer are discussed in the next subsection.

9.4.5 Effects of concrete capping above EPS sub-base

In this particular case a loadspreading concrete capping layer (E = 11,000 MPa) was placed above the EPS sub-base instead of the sand layer. The layer thickness of the concrete capping amounted to 0.1 m. Furthermore, it was assumed that no crack developed in
the capping layer above the open joint in the sub-base and that the effective stiffness in the unbound base material is not reduced, i.e. the elasticity modulus was equal to 500 MPa (instead of 200 MPa in the case without the concrete capping layer).

9.4.5.1 Resulting deformations

The calculated deformations, magnified with the identical magnification factor \((mf = 500)\) as in Figures 9.26 and 9.27, found in the motorway pavement with a concrete capping layer are illustrated in Figure 9.29. Contrary to Figure 9.27, the effects of the existing open joint in the sub-base are not noticeable.

![Figure 9.29](image)

**Figure 9.29**
Resulting deformations due to a 100 kN axle load in the model with a concrete capping layer above the EPS sub-base

9.4.5.2 Calculated deflections and strain values

Figure 9.30 shows the calculated deflections at the pavement's surface and asphalt strain values in the analyzed pavements with and without a concrete capping layer above the EPS sub-base. It is obvious that utilizing a concrete capping layer improves the overall performance of the pavement structures with EPS to a large extent. The identical asphalt strain values found below the left-hand and right-hand (wheel) loads suggest that the (negative) effects of the open joint(s) could be neutralized if a concrete capping layer is used.

The major conclusion that can be drawn by comparing Figures 9.30 and 9.28 is that implementation of the concrete capping layer enables the design and construction
of light-weight pavement structures with an EPS sub-base for the heavily loaded motorways. The design life is even longer than that of the traditional (heavy) structures with the sand sub-base.

**Figure 9.30** - Deflections at the pavement’s surface and the strain values at the bottom of the asphalt for the motorway models with a sand layer and a concrete capping layer respectively above the EPS sub-base

**9.5 CONCLUSIONS**

- Open joints between the EPS blocks in the sub-base significantly affect the behaviour of pavement structures with an unbound roadbase. The wide joints make it impossible to support properly the above-laid unbound base. This results in insufficient support of the roadbase to the asphalt layer. Consequently, higher stress and strain values occur in the asphalt layer under the wheel load with as final result a shorter life of the pavement structure.

- Implementation of a concrete capping layer above the EPS sub-base neutralizes the (negative) influence of the joints between the EPS blocks on the pavement behaviour. Such a capping layer ensures enough support to the unbound roadbase layers above this layer, also in the case of existing open joints between the EPS blocks.

- The maximum vertical stress values occurring in the EPS sub-base layers under the 100 kN standard axle load do not exceed the linear-elastic region experimentally determined for this material.
Application of a denser (and more costly) EPS type with a somewhat higher elasticity modulus in the sub-base has only a very limited influence on the horizontal strain values at the bottom of the asphalt layer and, therefore, on the overall behaviour of pavement structures with an EPS sub-base.

In the case of the polder roads the existence of open vertical joints in the EPS sub-base results into approximately 10% higher horizontal asphalt strains under the 100 kN standard axle loading when a wheel load is located above such a joint. Accordingly, if longitudinal open joints coincide with a wheel track it could lead to a reduction of the pavement life with about 40%.

EPS sub-bases where the blocks are laid in various patterns deform somewhat different under traffic load. Different sub-base behaviour occurs along the joints between the EPS blocks. However, the absolute deformation differences are so small that the effects on the structural pavement behaviour are of no practical importance.

The (negative) influence of the horizontal division of the sub-base into two sublayers seems to be very limited. Still, building-in the EPS blocks in at least two sublayers in the sub-base is recommendable. Avoiding open joints between the EPS blocks demands accurate laying and in order to do this it is easier to use less thick blocks because block dimensions always deviate somewhat.

In case of the polder road, pavement strengthening by increasing the asphalt layer thickness with 30% (extra 50 mm) or by building-in a 150 mm thick cement treated capping layer above the EPS sub-base appeared to be similarly beneficial with respect to the horizontal asphalt strain. The realized asphalt strain reduction amounts to approximately 30% resulting in a 6 times longer pavement life.

The horizontal strain values at the bottom of the asphalt layer of the motorway pavement structure with the sand sub-base are approximately 40% lower than the corresponding values above an open joint in the EPS sub-base. As a result of this the reduction of the pavement life would amount to about 12 times if the EPS sub-base has been built-in with a wide joint exactly along the wheel track.

The implementation of a concrete capping layer above the EPS sub-base enables the design and construction of light-weight pavement structures with an EPS sub-base, suitable for heavily loaded motorways. Its design life is even longer than that of the corresponding traditional structures with a sand sub-base.

9.6 REFERENCES

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CHAPTER 10

DESIGN GUIDELINES AND RECOMMENDATIONS

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10.1 INTRODUCTION

Generally speaking, the current Dutch design procedure for pavement structures with an EPS sub-base includes three steps. In the first step the list of requirements is being established by defining both the boundary conditions and the design starting points. In the next step it is checked whether the implemented light-weight sub-base assures sufficient reduction, if not elimination, of settlements without danger of movements due to buoyancy. Repeated calculations for different sub-base thickness values in the assumed pavement structure lead iteratively to the optimum EPS layer thickness. Finally the durability of the considered pavement structure and built-in materials is checked for the expected traffic intensity during the design life. Based on these calculations the thickness of the upper pavement layers above the EPS sub-base has to be determined. The flowchart of the pavement design procedure is illustrated in Figure 10.1 [1].

The boundary conditions are: in-situ subsoil conditions, groundwater level and traffic intensity. Important subsoil conditions are the geotechnical profile, the density and thickness of the layers, the sensitivity to settlements, the soil mechanical history (settlements in the past) etc. The starting points, i.e. arbitrarily defined preconditions, are: the pavement design life expressed as the number of 100 kN standard axle load repetitions; the street level; and pavement material characteristics of the previously selected type of construction.

This design procedure differs to a certain extent from the Dutch design procedure that is followed in the case of a sub-base of traditional materials (mainly sand for Dutch conditions). The differences regard the weight-balance and the buoyancy calculations. Both calculations serve the purpose of determining the proper thickness of the EPS sub-base. With "proper EPS sub-base thickness" is meant such a thickness that subsoil settlements are eliminated or reduced to an acceptable amount; water absorption in the EPS sub-base is taken into account in the subsoil stress calculation by assuming an EPS density of 100 kg/m³. The excavation depth for the EPS material may be restricted by considering the buoyancy forces for the case of highest possible ground water level. Even if areas are flooded the upper pavement layers must be heavy enough to keep the EPS sub-base
in position. In the buoyancy calculation the dry density of the EPS sub-base material is used. The minimum safety factor recommended for buoyancy calculations amounts to 1.1 [2].

Once the proper EPS layer thickness has been determined the design procedure continues with calculation of the pavement design life based on the Shell Pavement Design Manual [3]. This mechanistic procedure is the main pavement design method used in the Netherlands. The Shell Pavement Design Manual considers the horizontal tensile strain at the bottom of the asphalt layer and the vertical compressive strain at the top of the subgrade to be of critical importance for the design. The asphalt strain value has to be limited in order to prevent asphalt fatigue cracking while the limitation of the vertical strain serves to prevent excessive permanent deformation in the subgrade. In the Manual strain values are given as a function of the allowable number of load applications. So, by knowing the strain values, one is able to determine the pavement design life expressed as 'allowable number of 100 kN axle load repetitions'.

The missing part in the discussed current design procedure for pavement structures with an EPS sub-base is a design criterion regarding the EPS material. There is no established maximum value for either strain or stress occurring in the EPS layer due to the traffic load, the limit value which should not be exceeded because of negative effects on the material behaviour.

Cyclic loading test results presented in subsection 5.6.3 point out that EPS15 does not accumulate permanent deformations under combined static and cyclic stress of 15 kPa and 20 kPa respectively. The related total strain amounted to 0.6% (paragraph 5.6.3.2). EPS20 resisted a cyclic stress component of 30 kPa, i.e. undergoing cyclically a total strain of about 0.7%, without permanent deformation. The linear-elastic region for EPS15 and EPS20 may be assumed not to be lower that 0.4% according to the compression test results (subsection 5.4.1).

The strain occurring in the EPS layer is a result of the dead weight of the upper pavement layers, on one hand, and the traffic loading, on the other. Generally speaking, the higher the (static) strain component due to the dead weight, the lower the (dynamic) component due to the traffic loading. The static strain due to the dead weight of a thin pavement structure (where a relatively high dynamic strain can be expected) amounts to about 0.2% (subsection 5.4.3).

Based on the results reviewed above it may be stated that as long as the elastic deformation in the EPS sub-base due to repeated (traffic) loads is limited to 0.4%, then permanent deformation of the EPS blocks is negligible and will have no influence on the pavement performance. Therefore, the design criterion for the EPS layer should be a maximum strain value of 0.4%.

In completed pavement structures the strains in the EPS sub-base due the traffic loads are unlikely to be critical. More problems can be expected in the construction phase before all layers are built-in. If the EPS layer is overstressed by the construction traffic driving on (unbound) base layers, the effective EPS elasticity modulus is reduced (paragraph 5.5.4.1) and the water absorption increases.

Pavement analyses by means of both multi-layer [4] and finite element models (chapter 9) pointed out a negligible influence of the EPS thickness on the structural pavement
Design Guidelines and Recommendations

behaviour. Due to the low elasticity modulus the EPS block layer simply does not contribute to the load distribution and functions only as a fill material.

Since the stress and strain values in the pavement layers are independent of the thickness of the EPS sub-base it is possible to determine the pavement design life before carrying out settlement and buoyancy calculations. As input value an unit EPS thickness, e.g. 0.5 m, could be applied. The advantage of such an approach is that the upper pavement layers can be designed first and their total weight thus is known before carrying out the weight-balance calculation and determining the thickness of the EPS layer.

10.2 DESIGN GUIDELINES

Based on the considerations given in the previous chapters the following guidelines for the design of pavements with an EPS sub-base are given.

- In designing the pavement it must be realized that EPS20 blocks that are in contact with water will absorb about 2% v/v of water. EPS15 blocks will absorb more water, probably about 3% v/v. Although these volume percentages are low it means a considerable increase in weight which has to be taken into account when designing roads with EPS sub-bases. The usually assumed maximum density of saturated EPS of 100 kg/m$^3$ contains a high safety factor, a density of 40 to 50 kg/m$^3$ seems to be a more realistic value.

- During construction much attention should be paid to proper placement of the EPS blocks. All joints should be closed and load transfer across the joints should be promoted. As in the case with concrete block pavements, filling of the joints with jointing sand is strongly recommended. In order to be able to fill the joints a V type joint is recommended (Figure 10.2). Blocks with such joints can be easily made.

- Longitudinal joints between EPS blocks must not coincide with a wheel track because it will result into a significant reduction of the pavement life. The EPS block pattern should be designed such that longitudinal joints are located between the wheel tracks.
As long as the elastic deformation due to repeated (traffic) loads is limited to 0.4%, then permanent deformation of both EPS15 and EPS20 blocks is negligible and will have no influence on the pavement performance. Therefore, the vertical strain value of 0.4% should be used as the design criterion for the EPS layer.

The EPS thickness has a negligible influence on the pavement structural behaviour. Therefore, first the upper pavement layers should be designed by using an unit EPS thickness, e.g. 0.5 m, and then, when the exact dead weight of the upper pavement layers is known, the weight-balance calculations should be performed and the proper EPS thickness determined. The revised design procedure, including the $e_{EPS}$ criterion, is shown in Figure 10.3.

During the construction phase, which is the most critical phase, special measures (such a steel planking) should or could be taken to ensure that the maximum allowable EPS strain value of 0.4% is not exceeded. Overloading EPS results in a lower modulus of elasticity and a higher water absorption.

The presence of an EPS sub-base in a pavement structure has a significant influence on the stress and strain development in the pavement. If granular materials are placed immediately on top of the EPS layer then the stiffness of such a layer is low and in fact much lower than is normally expected. Consequently, unbound material modulus values reduced up to 50% should be used as input data for design purposes.

Unbound base materials that have the potential to develop a high stiffness do not pay off. Also relatively expensive self-cementing materials (e.g. blast furnace slags) seem to be not adequate above an EPS sub-base as cementing does not develop because of a significant amount of movement in the structure caused by the heavy traffic loads.

For design purposes a minimum elastic modulus of 5 MPa can be adopted for EPS20. In the case of EPS15 a minimum modulus of 4 MPa should be used.
Neither the modulus of EPS nor its other characteristics will deteriorate due to environmental influences like repeated wetting and freeze-thaw cycles.

The application of denser (and more expensive) EPS types with a somewhat higher elasticity modulus has no significant effects on the overall pavement behaviour. The use of EPS15, the lightest EPS type, instead of EPS20 can reduce the material costs considerably. However, one has to realize that the vertical strains in EPS15 will be about 1.25 times greater than those in EPS20, while the criterion $\varepsilon_{\text{EPS}} \leq 0.4\%$ is still valid.

Application of a cement treated capping layer on top of the EPS sub-base has a tremendously beneficial effect on the performance of the pavements. Such a capping layer neutralizes the effects of open joints between the EPS blocks, guarantees sufficient support to overlaying unbound base material even under high traffic intensity and eliminates any restriction for use of cheaper low-density EPS types. A cement treated capping layer is therefore strongly recommended.

Although additional work needs to be done to validate the test procedure modulus testing by means of the ultrasonic method is very promising for quality control of the blocks on site.

10.3 RECOMMENDATIONS

Resulting deformations in 3-D pavement models indicate that reinforcement of the granular base course could contribute to a better overall performance of pavement structures with only an unbound base above the EPS sub-base (in accordance with suggestions in [5, 6]). By means of a reinforcement, lateral confinement develops which results in a higher base resilient modulus. The development of interface elements allow analyses of pavement structures with such reinforced bases, and this kind of analysis is strongly recommended. This is especially the case since reinforcement of the granular base could be a very cost effective solution.

More work has to be done in order to validate the results obtained by means of the ultrasonic testing method. A modification of the existing equipment is necessary so that the ultrasonic apparatus can be fully applicable for testing of large EPS blocks on construction sites.

From this research program it is quite obvious that consultants and contractors involved in road projects containing EPS sub-bases should be very well aware of the mechanisms that control the performance of such pavements. The development of training courses to be given by industry together with the Delft University is therefore strongly recommended.
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CHAPTER 11

CONCLUSIONS
CONCLUSIONS

As elaborated at the beginning of the thesis (section 1.2) five objectives were defined for the research program: a. inventory of potential light-weight sub-base materials and characterisation of foamed concrete (a possible alternative material for EPS geofoam); b. characterisation of EPS20 and EP15 mechanical behaviour under sub-base conditions; c. characterisation of pavement structures in which an EPS sub-base is embedded; d. characterisation of in-situ pavements with an EPS sub-base; and e. definition of design guidelines for pavements with built-in EPS blocks. In Chapter 10 design guidelines and recommendations related to the last objective e. have already been discussed. Below the conclusions regarding the first four research objectives are given.

With respect to light-weight sub-base materials the following general statement can be made:

a.1 The purpose of the use of light-weight fill/foundation materials in sub-bases is to obtain a significant reduction of the deadweight of a pavement structure in a cost effective way. Having in mind the representative wet density of compressible soil actually only expanded polystyrene (EPS) geofoam and foamed concrete fulfill the requirements of light-weight materials on a large scale.

As to foamed concrete, which is to some extent an alternative for EPS geofoam, the conclusions drawn from literature study and material testing are:

a.2 Of all foamed concrete mixes with densities between 400 and 1600 kg/m³ only the lightest foamed concrete types (with a density up to 600 kg/m³) are sufficiently lighter than the subsoil to enable the construction of low-settlement pavement structures in areas with weak soil conditions.

a.3 By taking into account the long-term (negative) effects of cracks developing due to shrinkage and thermal action and an aggressive environment (acid groundwater), the maximum water absorption of the mixes with densities of 400 and 600 kg/m³ during the pavement design life (usually a 20-years period) is estimated to reach up to 15% and 10% by volume for foamed concrete sub-bases laid below the groundwater.

a.4 Experimentally determined E-values under cyclic compressive loading of the foamed concrete mix with a density of 500 kg/m³ were lower than the values reported in (Dutch) literature. Under the maximum vertical stress values expected in (thin concrete block) pavement structures, the experimentally determined moduli ranged between 400 and 500 MPa which is more than 20% lower than the reported E-values.

a.5 The maximum vertical stress values occurring in the (properly hardened) foamed concrete sub-base hardly exceed 30% of the material (four-weeks) compressive strength. Therefore the design criterion regarding the maximum allowable vertical stress will be practically always satisfied.
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a.6 There is a wide dispersion of the maximum cyclic tensile stress levels which foamed concrete can sustain before cracks occur, probably due to presence or absence of either micro cracks or weak spots at the location of stress concentration. However, the tests indicated that at stress level lower than 115 kPa there is no need to worry about fatigue.

Summarising the experimental results of the materials research on EPS20 and EPS15 it can be concluded that:

b.1 EPS20 absorbs water very slowly and to a limited extent. The asymptotic value of 2% v/v can be assumed as the maximum percentage of water which EPS20 will absorb. Corresponding asymptotic water absorption of EPS15 is about 3% v/v.

b.2 The dominant transport mechanism responsible for water absorption seems to be diffusion. Initiated by a vapour concentration difference, vapour penetrates progressively through the thin walls of the cells. Further diffusion into the inner layer cells is more difficult.

b.3 It seems that after the cell structure is damaged due to overloading, water penetrates faster into the EPS and accumulates not only in small voids between the fused cells but in the collapsed cells as well. This causes an increase of the maximum water content in the EPS when deformed beyond the failure limit of its cell structure.

b.4 Low temperatures, water absorption level and exposure to freeze-thaw cycles, separately or combined, seem to have no negative influence on the mechanical behaviour of EPS. (The modulus of elasticity for the EPS geofoam in wet state at low temperatures will not be lower than in dry state at normal temperature.)

b.5 The linear-elastic region for EPS20 under slow compressive loading at both normal and low temperatures is characterized by a boundary strain values, $\varepsilon_{\text{in}}$, in the range of 0.4% up to 0.6%. The corresponding boundary stress values vary between 40 and 50 kPa. The E moduli vary between 6.2 and 12.4 MPa.

b.6 There is a significant reduction in resistance to deformations of EPS15 and EPS20 when the material is loaded beyond its linear-elastic limit. Such a change in EPS behaviour does not proceed gradually but dramatically once its cell structure is damaged. In practice it implies that overloading of the EPS sub-base would lead to a lower $E_{\text{EPS}}$ value in service, which would be lower than the input $E_{\text{EPS}}$ value used for pavement design. The load carrying capacity is maintained beyond the elastic limit.

b.7 Under a static stress of 15 kPa combined with a cyclic stress lower or equal to 20 kPa EPS15 seems to behave elastically. Accompanying elastic cyclic strain (average value representative for a whole EPS15 block) amounts about 0.4% while the total strain value stays within 0.7%. The corresponding dynamic modulus of elasticity of EPS15 blocks stays constant with a minimum $E_{\text{dyn}}$ value of 5.5 MPa. Significant permanent deformation will develop in the EPS15 blocks built-in in a pavement structure if a cyclic stress component caused by traffic load has values of 35 kPa or higher.

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b.8 The resistance to permanent deformation of the blocks of EPS20 is significantly higher compared to EPS15. A cyclic stress of 30 kPa combined with a static stress of 15 kPa does not cause any permanent strain of significance. Corresponding cyclic strain (average value for a whole EPS20 block) has a constant value of 0.34% while accompanying total strain amounts 0.6%, which means better stress-strain behaviour of EPS20 under $\sigma_c = 30$ kPa compared to EPS15 under $\sigma_c = 20$ kPa. The $E_{dyn}$ value of EPS20 blocks under considered stress conditions is approximately 9.0 MPa. The $E$ values are somewhat larger than the value of 5 MPa which is normally used in pavement design procedures.

b.9 Combined static and dynamic (cyclic) high level loads which could exceptionally occur in an EPS sub-base can contribute to the development of a certain low permanent deformation in EPS. However, even a large number of load repetitions of the highest expectable cyclic load does not result in an increase of the cyclic deformations. This points out that excessive permanent deformation is not likely to occur in EPS20 in various states under extreme stresses which can be expected in the sub-base. Significant vertical permanent deformation occurs in EPS20 blocks exposed to a 15 kPa static stress, when cyclic stress component amounts to 50 kPa or more.

b.10 Total vertical permanent strain in the EPS sub-base due to the deadweight of the toplayers has a range of approximately 0.5% in the case of long-term loading due to very heavy pavement structures (corresponding static stress of about 20 kPa). The immediate vertical strain in EPS blocks, a significant part of the total vertical strain, amounts to approximately 0.3%. The remaining vertical strain of 0.2% is the result of creep after more than a year. About half of the expected maximum creep in the EPS occurs already within the first day. Therefore, the additional permanent vertical deformation of the pavement structure caused by creep of the EPS sub-base layer is of minor practical importance for pavement design.

b.11 The Poisson's ratio values of EPS20, calculated from cyclic radial and vertical strains under realistic sub-base stress conditions, varied between 0.07 and 0.11.

b.12 The ultrasonic test method has a potential to be used on site to determine the density of various positions in EPS blocks for quality control purposes. This is made possible by the close relationship between the density and the elastic modulus of EPS. More work has to be done in order to validate the results obtained by means of the ultrasonic testing method. Also, modification of the existing equipment is necessary so that the ultrasonic apparatus can be fully applicable for testing of large EPS blocks on construction site.

The finite element analyses of temperature distribution in asphalt pavement structures with an EPS sub-base lead to the next conclusions:

c.1 The use of EPS geofoam in the sub-base affects the temperature distribution in pavement structures but the asphalt temperatures have practically the same values as in the corresponding pavement structures with common (sand) sub-bases. The EPS geofoam is at too great depth to cause any temperature difference in the asphalt layers. This means that the use of EPS in asphalt pavement structures does
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not require to take into account a different asphalt temperature for determination of the pavement life.

c.2 When applying an EPS sub-base, the frost penetration depth is greater than in case of a sand sub-base. However, in normal construction practice the frost penetration (which in the Netherlands rarely exceeds 1 m) will not reach below the sub-base, which means that the EPS foam will also not lead to a lower bearing capacity problem of the (peat) subgrade as a result of freeze-thaw periods.

Based on the results obtained from the non-linear finite element analyses as well as from the 3-D finite element analyses on pavement structure models with implemented joints between the EPS blocks in sub-bases, the following conclusions are drawn:

c.3 The horizontal strain at the bottom of a (200 mm thick) asphalt layer is approximately 15% higher when an EPS sub-base is built-in below an unbound roadbase instead of sand. Such a strain difference results in an approximately two times lower allowable number of standard axle load repetitions, i.e. a two times shorter pavement design life if an EPS sub-base is applied.

c.4 The thickness of the EPS sub-base has only a marginal influence on the pavement behaviour under loading and on the pavement design life. The stress and strain values in the upper pavement layers are almost the same in pavement structures with different thicknesses of the EPS layer.

c.5 Unbound roadbase materials have a higher 'effective' stiffness in pavement structures in case a cement-bound load-spreading layer is applied above the EPS sub-base. Therefore, the assumed E-value for the unbound material in linear-elastic calculations should be lower when this material is laid directly over an EPS sub-base than in the case of the presence of an intermediate cement bound layer.

c.6 Implementation of a cement-treated (concrete) capping layer on top of the EPS sub-base substantially increases the design life of the pavement structure with an EPS sub-base. The application of such a layer is therefore recommended for pavement structures subjected to heavy traffic loading.

c.7 Maximum vertical and horizontal stress and strain values calculated in the EPS layer of all the analyzed pavements due to a standard 100 kN axle load do not exceed the experimentally determined boundaries of the elastic region of the EPS.

c.8 Open joints between the EPS blocks in the sub-base significantly affect the behavior of pavement structures with an unbound roadbase. The wide joints make it impossible to support properly the above-laid unbound base. This results in insufficient support of the roadbase to the asphalt layer. Consequently, higher stress and strain values occur in the asphalt layer under the wheel load with as final result a shorter design life of the pavement structure.

c.9 Implementation of a concrete capping layer above the EPS sub-base neutralizes the (negative) influence of the block joints on the pavement behavior. Such a capping layer ensures enough support to the unbound roadbase layers above this layer also in the case of existing open joints between the EPS blocks.
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c.10 In the case of the polder roads the existence of open vertical joints in the EPS sub-base results into approximately 10% higher horizontal asphalt strains under the 100 kN standard axle loading when a wheel load is located above such a joint. Accordingly, if longitudinal open joints coincide with a wheel track they could contribute to a reduction of the pavement design life of about 40%.

c.11 In case of the polder road pavement strengthening by increasing the asphalt layer thickness with 30% (extra 50 mm) or by building-in a 150 mm thick cement treated capping above the EPS sub-base appeared to be similarly beneficial with respect to the horizontal asphalt strain. The realized asphalt strain reduction amounts to approximately 30%.

c.12 The horizontal strain values at the bottom of the asphalt layer of the motorway pavement structure with a sand sub-base are approximately 40% lower than the corresponding values above an open joint in the EPS sub-base. The consequential reduction of the pavement design life would amount to more than ten times if the EPS sub-base has been built-in with a wide joint exactly along the wheel track.

c.13 The implementation of a concrete capping layer above the EPS sub-base enables the design and construction of light-weight pavement structures with EPS suitable for heavily loaded motorways, of which the design life is even longer than that of the corresponding traditional structures with a sand sub-base.

c.14 EPS sub-bases where the blocks are laid in various patterns deform differently under traffic load. Different sub-base behaviour occurs along the joints between the EPS blocks. However, the absolute deformation differences are so small that the effects on the structural pavement behaviour are of no practical importance.

c.15 The (negative) influence of the horizontal division of the sub-base into two sublayers seems to be very limited. Still, building-in of the EPS blocks in at least two sublayers in the sub-base is recommendable. Avoiding open joints between the EPS blocks demands accurate laying and in order to do this it is easier to use less thick blocks.

c.16 Application of a denser (and more costly) EPS type with a somewhat higher elasticity modulus in the sub-base has only a very limited influence on the horizontal strain values at the bottom of the asphalt layer and so on the overall behaviour of the pavement structures with the EPS sub-base.

The periodical in-situ measurements on concrete block pavement structures with EPS sub-bases led to the following conclusions:

d.1 The deflections on concrete block pavement structures with an EPS sub-base are significantly bigger than the deflections on classic block pavements with a sand sub-base. This can be explained by the lower E-modulus of the used EPS compared with a sand sub-base.

d.2 The critical positions are often located in the cross-sections with drains on both sides of the road. The local deadweight of the pavement structure is higher because part of the EPS layer is replaced by the drains. All this means that cross-sections
Conclusions

with drains should be carefully designed and constructed (compaction aspects) to avoid the local problems on the block pavement structures with an EPS sub-base.

d.3 A modulus of elasticity of 5 MPa, which is usually assumed for EPS15 and EPS20, seems to be a proper value for the design of block pavement structures. The back-calculated $E_{EPS}$ has a value from 4 up to 12 MPa which means an E-value of 5 MPa should be safe enough for use in practice.

d.4 The results of the back-calculation analyses performed on average and characteristic profiles point out a big influence of the effective stiffness of the unbound material on the general pavement behaviour. It also emphasizes the importance of a proper compaction of the unbound roadbase materials in structures with an EPS layer.

d.5 The back-calculated elasticity moduli for the Blast Furnace Slags (BFS) roadbase amounted to less than 50% of the potential values for this material. The expected self-cementing apparently did not occur in the BFS at all. Therefore, the choice of the relatively expensive BFS should be critically reviewed because in this case its effective stiffness does not exceed those of cheaper materials.

d.6 Driving-in thin profiled concrete tiles in a weak road verge is not an adequate substitute for heavy kerb stones. This is because the thin concrete tiles cannot provide the needed lateral support to concrete block pavement structures. Insufficient lateral support results not only in wide joints and ineffective load transfer between the blocks but also to a certain decrease in the effective stiffness of the lower built-in roadbase layer. The final result is the rapid occurrence of severe ruts in the wheel track on sections where the concrete tiles have been used.

From the results obtained from falling weight deflection measurements and asphalt strain measurements on a particular asphalt pavement structure with a built-in EPS layer, it can be stated that:

d.7 The very low back-calculated E-values for the sand (from 40 to 65 MPa) and the crushed masonry/concrete base (from 80 to 85 MPa) before overlaying highlight the inability of the EPS to provide a proper support to the roadbase in the considered pavement structure with a 130 mm thick asphalt layer. Correspondingly insufficient support of the roadbase to the asphalt layer resulted in a critically high asphalt strain of about 192 $\mu$m/m ($T=20^\circ$C). Use of overestimated E-values for the roadbase materials was probably the main reason for the inappropriate pavement design.

d.8 Open joints between the EPS blocks in a sub-base can have very serious consequences for the design life of pavement structures, and thus have to be avoided by all means. The longitudinal joints between the EPS blocks should not be close to a wheel track. An adequate (lateral) support of the blocks is necessary to prevent any movement of the blocks.

d.9 The back-calculated E-value of the EPS sub-base ranges from 10.4 to 19.7 MPa, which is somewhat higher than the elasticity moduli found in literature for the EPS types under consideration (EPS25 and EPS30).
Conclusions

At the end, as the final conclusion of the thesis, it may be stated that the use of EPS makes it possible to design and construct of the light pavement structures that give good performance provided that the guidelines given in Chapter 10 are taken into account.
Appendix I

Foamed concrete mix constituents

<table>
<thead>
<tr>
<th>Producer: VOORBIJ GROEP WILNIS Departement R&amp;D</th>
<th>Code: DP500/16.12.03 BVK/-</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date:</td>
<td>March 24, 1995</td>
</tr>
<tr>
<td>Regarding:</td>
<td>Test 95-13</td>
</tr>
<tr>
<td>Reference:</td>
<td>R501</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Foamed concrete slurry components</th>
<th>Mass [kg]</th>
<th>Density [kg/m³]</th>
<th>Abs. volume [10⁻³ m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement ENCI Pcb (42,5R)</td>
<td>160</td>
<td>3,150</td>
<td>50.8</td>
</tr>
<tr>
<td>Foaming agent V₁₀</td>
<td>120</td>
<td>2,250</td>
<td>53.3</td>
</tr>
<tr>
<td>Water</td>
<td>160</td>
<td>1,000</td>
<td>160.0</td>
</tr>
<tr>
<td>Filler limestone dust</td>
<td>30</td>
<td>2,750</td>
<td>10.9</td>
</tr>
<tr>
<td>PROVOTON foam 3% solvent</td>
<td>30</td>
<td>42</td>
<td>722.0</td>
</tr>
<tr>
<td>Foamed concrete slurry</td>
<td>500</td>
<td>-</td>
<td>1,000</td>
</tr>
</tbody>
</table>

Table I.1 - Details about foamed concrete constituents of the mix tested in the scope of the testing program described in section 4.6
Appendix II

Details about specimens material and its production process

**PROCESS - PRESSURES - TEMPERATURES**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>pre-steam vacuum</td>
<td>0.60</td>
<td>0.00</td>
<td>foam pressure max.</td>
<td>1.60</td>
<td>0.03</td>
</tr>
<tr>
<td>steam with vacuum</td>
<td>0.15</td>
<td>0.00</td>
<td>foam pressure min.</td>
<td>0.05</td>
<td>0.03</td>
</tr>
<tr>
<td>steam pressure</td>
<td>0.80</td>
<td>0.00</td>
<td>reserve</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>preheat T FII</td>
<td>95</td>
<td>70</td>
<td>preheat T NOW</td>
<td>95</td>
<td>72</td>
</tr>
</tbody>
</table>

**SET PRESSURES**

| | side wall FII | 0.00 | 0.74 | side wall NOW | 0.00 | 0.73 |
| | bottom | 0.00 | 0.76 | lid | 0.00 | 0.76 |

**PROCESS - TIMES**

<table>
<thead>
<tr>
<th>time (sec)</th>
<th>set</th>
<th>act.</th>
<th>time (sec)</th>
<th>set</th>
<th>act.</th>
</tr>
</thead>
<tbody>
<tr>
<td>fill</td>
<td>5.0</td>
<td>0.0</td>
<td>cross steam 2</td>
<td>3.0</td>
<td>0.0</td>
</tr>
<tr>
<td>purge mould</td>
<td>12.0</td>
<td>0.0</td>
<td>autoclave</td>
<td>10.0</td>
<td>0.0</td>
</tr>
<tr>
<td>divert condensate</td>
<td>6.0</td>
<td>0.0</td>
<td>pressure maintain</td>
<td>2.0</td>
<td>0.0</td>
</tr>
<tr>
<td>pre-steam vacuum</td>
<td>60.0</td>
<td>0.0</td>
<td>press. reduct. steam</td>
<td>6.0</td>
<td>0.0</td>
</tr>
<tr>
<td>steam bottom-lid</td>
<td>10.0</td>
<td>0.0</td>
<td>press. reduct. foam</td>
<td>60.0</td>
<td>0.0</td>
</tr>
<tr>
<td>cross steam 1</td>
<td>5.0</td>
<td>0.0</td>
<td>preheat autoclave</td>
<td>12.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

**BAR CHARTS**

- **press. decay time**: 924.4 sec
- **cycle time**: 1119.2 sec
- **date**: 12:04:91
- **time**: 09:07:21
- **block no.**: 600

**Steam 0.80**

- 0.0
- 0.2
- 0.4
- 0.6
- 0.8
- 1.0
- 1.2
- 1.4
- bar

**Temp. A 120**

**Temp. B 117**

**Steam quant. 8.2**

- 6.0
- 16.0
- 16.0
- 20.0
- 25.0
- 30.0

250
CURRICULUM VITAE

Family name: Duškov
Forename: Milan
Date of birth: November 10, 1960
Place of Birth: Belgrade, Yugoslavia

Academic degrees

B.Sc. Degree at Faculty of Civil Engineering of Belgrade University in 1987
M.Sc. Degree at Faculty of Civil Engineering of Delft University of Technology in 1989

Professional career

1989 - 1996 Research Engineer at the Road en Railroad Research Laboratory of the Delft University of Technology, The Netherlands
1996 - ... Consultant Pavement Engineering at Oranjewoud B.V. in Capelle a/d IJssel, The Netherlands