THE UPGRADED DUTCH DESIGN METHOD FOR CONCRETE BLOCK ROAD PAVEMENTS

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ABSTRACT

Small concrete elements are widely used in the Netherlands, mainly concrete blocks 80 mm thick and horizontal dimensions of 211 mm x 105 mm. In industrial areas the thickness of the blocks is increased to 100 mm or 120 mm. When only minor traffic loads are expected blocks with a thickness of 70 mm are occasionally applied.

Due to the fact that the quality of the subgrades in the Netherlands is poor, sand is regarded as a structural material for roads. Higher-quality natural materials are simply not available. As a result, sand is often used as a sub-base in concrete block pavements. If a base is needed, it is usually a mixture of crushed masonry and crushed concrete. Of these recycled materials at least 50% (m/m) should be crushed concrete.

It is known from both practice and laboratory testing that the behaviour of sands is heavily dependent on their source. The degree of compaction is also a very determining factor.
A universal design method for Dutch concrete block pavements should thus take into consideration the type of sand that is to be used in the sub-base and its degree of compaction. Similar information about the base should also be taken into account.

In this paper, the development of a universal design method for concrete block pavements in The Netherlands is discussed and major findings presented. The method is largely based on a mechanistic determination of rut development. This mechanistic approach requires the mechanical properties of all the materials used in the substructure. For this reason, equations describing the mechanical properties of the materials involved as a function of material composition, grading, grain shape and compaction are derived by regression analysis. Their development results in an improved understanding of the behaviour of unbound materials used in the Netherlands. As such, and combined with the mechanistic approach of rutting, they enable the development of a new design method.

This paper only describes the behaviour of concrete block pavements without a base, so with a sand sub-base only. For this type of structure the design method is explained and design examples illustrate the relative effect of various design parameters.

1. INTRODUCTION

Small concrete elements are widely used in the Netherlands, mainly concrete blocks 80 mm thick and horizontal dimensions of 211 mm x 105 mm.

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In industrial areas the thickness of the blocks is increased to 100 mm or 120 mm. When only minor traffic loads are expected blocks with a thickness of 70 mm are occasionally applied.

The recent trend in the Netherlands towards also using blocks in architectural applications has led to the use of concrete elements with a shape and thickness that differs from the above standards. These elements are also available in a large variety of colours.

Clearly, then, any design procedure for concrete block pavements in the Netherlands must consider the dimensions of the block itself.

Due to the fact that the quality of the subgrades in the Netherlands is poor, sand is regarded as a structural material for roads. Higher-quality natural materials are simply not available. As a result, sand is often used as a sub-base in concrete block pavements. If a base is needed, it is usually a mixture of crushed masonry and crushed concrete. Of these recycled materials at least 50% (m/m) should be crushed concrete.

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A universal design method for concrete block pavements should thus take into consideration the type of sand that is to be used in the sub-base and its degree of compaction. Similar information about the base should also be taken into account.

In the Netherlands the official failure criterion for concrete block pavements is a 15 mm rut depth. However this criterion is hardly ever respected and larger rut depths, dependent on the situation, are accepted. This implies that there is a need for a design tool in which the failure criterion may be varied.

Apart from information about the materials used in the sub-base and base, the type of blocks and the need for a flexible failure criterion, a universal design method should also take into account the stiffness of the subgrade and the design load (magnitude and lateral wander). In this paper the development of a universal design method for concrete block pavements in The Netherlands is discussed. This work is the result of an ongoing tradition in research into concrete block pavement behaviour by the Delft University of Technology.

2. APPROACH

In 1988 the first Dutch design method for concrete block pavements (cbp) was presented in Rome at the Third International Conference on Concrete Block Paving (Houben, 1988) This method was based on test section behaviour combined with structural calculations.

In order to verify the method a research program was started in 1990 which ran until 1997. It resulted in a finite element model for cbps in which the behaviour of the unbound materials in the base and sub-base was addressed. The finite element code that was developed as part of the research is discussed extensively elsewhere (Huurman, 1996a; 1996b; 1997).

During the research program, the mechanical properties of eight sands and four base materials were also investigated. The material models that were applied are now briefly discussed.

This paper only describes the behaviour of cbps with a sand sub-base, which is the most widely used cbp structure in the Netherlands. A similar approach for cbps with an unbound base and a sand sub-base, although not discussed in this paper, has just been completed.

2.1 Strength

The strength of unbound materials is evaluated in terms of the Mohr-Coulomb criterion from which expression (1) was derived.

$$\sigma_{1,f} = \frac{(1+\sin\phi)\cdot\sigma_{conf} + 2c\cdot\cos\phi}{1-\sin\phi} \quad (1)$$

where

c:	cohesion [kPa]
Φ:	angle of internal friction [degr]
σ_{conf}:	confining pressure [kPa]
$\sigma_{1,f}$:	first principal stress at failure [kPa]

On the basis of (1) the stress ratio $\sigma_1/\sigma_{1,f}$ can be determined. This stress ratio increases with increasing material loading and reaches the value of 1 at failure.

2.2 Resilient Behaviour

For sands the following equation was found to give a proper description of the measured resilient modulus as a function of stress.

$$Mr = k1\left(\frac{\sigma_{conf}}{\sigma_0}\right)^{k2}\left(1-k3\left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{k4}\right) \quad (2)$$

where

Mr:	resilient modulus [MPa]
$\sigma_1/\sigma_{1,f}$:	stress ratio [-]
σ_{conf}:	confining pressure [kPa]
σ_0:	reference stress of 1 [kPa]
k1:	model parameter [MPa]
k2, k3, k4:	model parameters [-]

2.3 Permanent Deformation

The development of permanent deformation in sands was described by the following equation.

$$\varepsilon_{perm} = A\left(\frac{N}{1000}\right)^{B} \text{ with } A = a1\left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{a2} \text{ and } B = b1\left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{b2} \quad (3)$$

where

ε_{perm}:	permanent strain [%]
a1:	model parameter [%]
a2, b1, b2:	model parameters [-]

The research showed that the rutting behaviour of cbps heavily depends on the mechanical behaviour of the materials used in the substructure (i.e. bedding layer, base and sub-base).

Unfortunately, the mechanical behaviour of the materials used in the test sections that formed the basis of the Dutch design method for cbps (Houben, 1988) was not known. This, combined with the fact that the behaviour of cbps is heavily dependent on mechanical material behaviour, meant that it was not possible to completely validate the Dutch design method for cbps.

Given the importance of the mechanical behaviour of the unbound materials used in the substructure another research project commenced in 1997 and ran until 2002.

The main emphasis of this project was on the mechanical behaviour of sands and base materials. During this investigation the mechanical properties of sands were determined by triaxial testing. The degree of compaction, the grading and the grain shape of sands varied strongly. For base materials the degree of compaction, the grading and the ratio between crushed concrete and crushed masonry was varied. Also for base materials the mechanical properties were determined by extensive triaxial testing. The results of this research are extensively discussed in (Niekerk van, 2002) and not further discussed here.

In the development of a universal Dutch design method for concrete block pavements the results of the three research projects discussed above were combined.

The approach was as follows:
- Establish relationships between the mechanical behaviour of unbound materials and simple properties such as grading and grain shape.
- Create a data set of computations using the 1997 FEM model. In this data set the properties of the cb-layer, the materials in the substructure, the thickness of the various structural layers, the stiffness of the subgrade and the traffic loading should vary.
- Analyse the data set using regression, and so describe cbp behaviour for the purpose of a new design tool.

In the sections hereafter the outcome of the above process is briefly discussed for structures with a sand sub-base only.

3. MECHANICAL BEHAVIOUR OF SANDS

3.1 Compaction

As discussed in (Niekerk van, 2002), the degree of compaction was found to be one of the most important factors influencing the mechanical behaviour of both sands and base materials. A general relation that converted the degree of compaction to the quality of compaction, qc, was therefore derived. The use of this equation results in a qc-value of 1 at a DOC of 101%, which is the Dutch standard for base material compaction (CROW, 2000) (see Figure 1). It was arbitrarily decided that the maximum value of qc, qc1, should be 1.5.

$$qc = qc1 \cdot \exp\left(\left(-\left(\frac{rot}{DOC}\right)^{qc2}\right)\right) \quad \text{with} \quad rot = 101\% \cdot \ln(qc1)^{\left(\frac{1}{qc2}\right)} \tag{4}$$

where

qc:	quality of compaction [-]
qc1:	model parameter = 1.5 [-]
qc2:	model parameter = 8 [-]
DOC:	degree of compaction [%]
rot:	point of rotation [%] (i.e. the DOC at which qc equals qc1/e =0.552 [-])

In figure 1 the relation between qc and DOC is given.

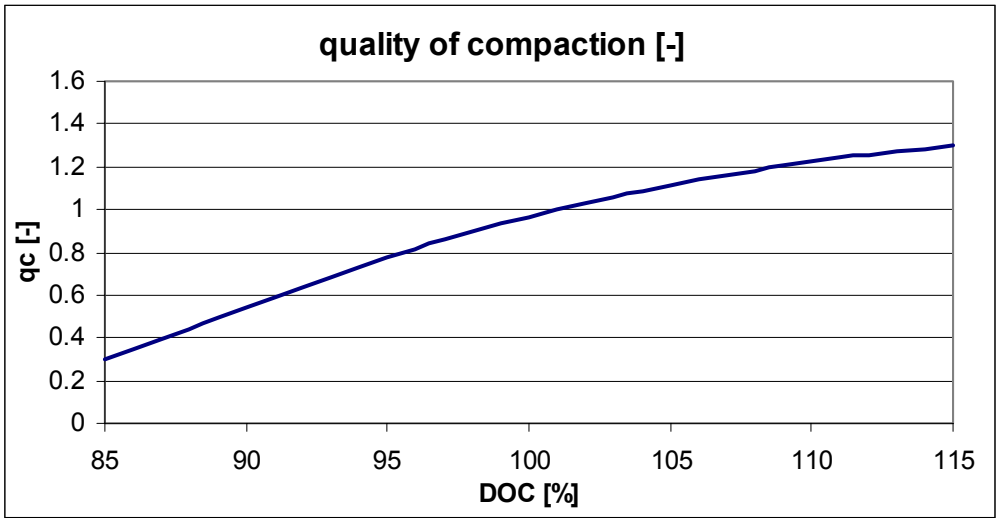

Figure 1. Quality of compaction as a function of the degree of compaction.

3.2 Strength

From (Niekerk van, 2002) the following strength data was obtained for sands typically used in the Netherlands, see Table 1.

Table 1. Properties of sands typically available in The Netherlands.

Sand	VVS [%]	Cu [-]	d50 [mm]	DOC [%]	c [kPa]	ϕ [degr.]
GR	63.4	2.45	0.227	97	10	39.2
KB	63.5	3.99	0.575	97	6.8	39
IJ-F	50	1.55	0.269	97	8.7	34.4
EM	54	1.69	0.239	97	1.4	41.4
WS	51.3	1.57	0.146	97	10	37.4
IJ-M	47	2.05	0.205	97	9.5	36.7
GR	63.4	2.45	0.227	100	18	41.7
KB	63.5	3.99	0.575	100	19.6	40.5
IJ-F	50	1.55	0.269	100	1.9	42.2
EM	54	1.69	0.239	100	6.8	40.2
WS	51.3	1.57	0.146	100	5.1	42.4
IJ-M	47	2.05	0.205	100	12.3	37.5
GR	63.4	2.45	0.227	103	25.1	42
KB	63.5	3.99	0.575	103	19.2	40.4
IJ-F	50	1.55	0.269	103	1.4	41.9
EM	54	1.69	0.239	103	5.8	44.7
WS	51.3	1.57	0.146	103	12.4	41.5
IJ-M	47	2.05	0.205	103	11.9	42.3

Sand: abbreviation of the location where the sand was found (source)

VVS: shape of the grains [%], 0%=glass pearls, 100% = standard crusher sand
Cu: coefficient of uniformity =d60/d10 [-]
d50: median grain size [mm]
DOC: degree of compaction [%]

From the analysis of the data in table 1 the following equations for c and Φ were derived.

$$c = c1 \cdot qc^{c2} \cdot \left(\frac{d50}{1mm}\right)^{c3} \cdot Cu^{c4} \cdot \left(\frac{VVS}{100\%}\right)^{c5} \quad \text{and} \quad \phi = \phi 1 \cdot qc^{\phi 2} \cdot \left(\frac{VVS}{100\%}\right)^{\phi 3} \tag{5}$$

where

c1: model parameter = 0.1375 [kPa]
c2: model parameter = 2.553 [-]
c3: model parameter = -1.698 [-]
c4: model parameter = 2.959 [-]
c5: model parameter = 0.384 [-]
Φ1: model parameter = 45.71 [degr]
Φ2: model parameter = 0.833 [-]
Φ3: model parameter = 0.091 [-]

Equation 1 may be used to determine the strength of a sand, $\sigma_{1,f}$, as a function of c, Φ and σ_{conf}. The fit between the strength obtained from the measured c and Φ values (see Table 1) and the strength obtained from the modelled c and Φ values (see equations 5) is given in Figure 2. In this Figure, the strength of a sand, $\sigma_{1,f}$, as explained by the data in Table 1 is plotted on the horizontal axis whilst the strength determined using equation (5) is plotted on the vertical axis. The figure shows the fit for a confining pressure, σ_{conf}, of 5 kPa and 25 kPa.

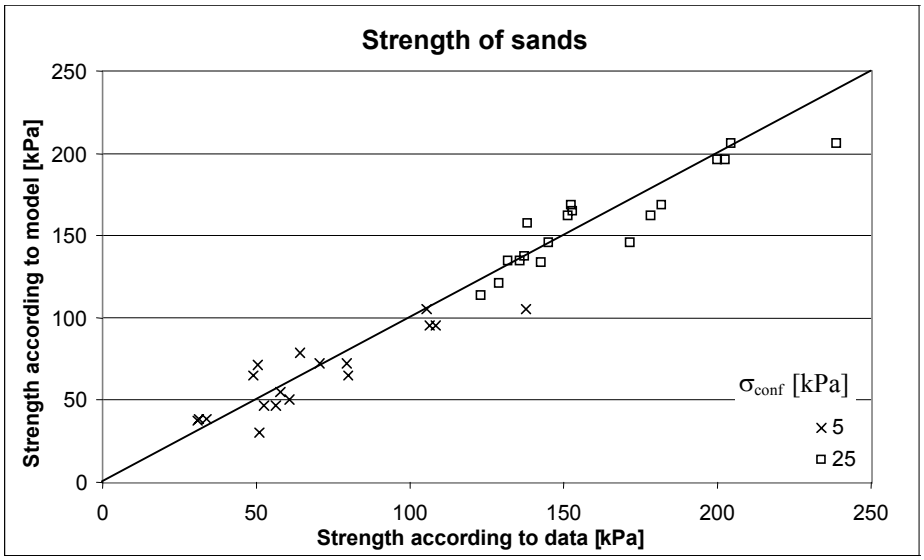

Figure 2. Fit between modelled strength of sands and strength on basis of material research.

3.3 Resilient Behaviour

As there was no data available about the resilient behaviour of sands from (Niekerk van, 2002), data from the earlier study (4) was used. Typical results are presented in Table 2.

Table 2. Typical Resilient Behaviour Properties of Sands.

Sand	VVS [%]	Cu [-]	d50 [mm]	DOC [%]	k1 [MPa]	k2 [-]	k3 [-]	k4 [-]
ZW	72.13	1.80	0.24	100	30.42	0.59	0.47	7.68
WE	72.85	1.68	0.24	100	38.4	0.6	0.89	5.79
AL	66.52	1.87	0.24	100	45.95	0.52	0.36	7.05
CB	94.11	2.17	0.27	100	47.75	0.58	0.67	1.51
CR	115.74	10.44	1.75	100	19.23	0.54	0.19	8.63
BA	74.29	1.86	0.15	100	44.96	0.52	0.28	5.83
PA	66.59	2.46	0.27	100	34.43	0.69	0.37	4.97
MH	72.4	4.00	0.23	100	38.64	0.68	0.42	4.04

Equations to determine the values of the parameters k1, k2, k3 and k4 were derived and these are presented in Appendix A.

In terms of explaining the behaviour of cbps the parameters k1 and k2 proved to be the most useful. These parameters describe the stiffness of sands when they are not heavily stressed (i.e. at low stress ratios, $\sigma_1/\sigma_{1,f}$).

Figure 3 compares the stiffness of the sands at minor stress ratios (when only k1 and k2 are of importance) predicted by the equations given in appendix A and and the results of material testing. As indicated in the Figure, three levels of σ_{conf} (i.e. 5, 20 and 100 kPa) were considered. The sum of squares was minimised in the regression process. Since sands mostly fail due to a lack of confinement, a weighting factor of 3 was applied to the sum of squares obtained at 5 kPa confinement stress. Weighting factors of 2 and 1 were applied for the 20 kPa and 100 kPa confinement stresses respectively. The effects of these weighting factors is reflected in Figure 3 which shows that the best fit was obtained at the 5 kPa confinement stress.

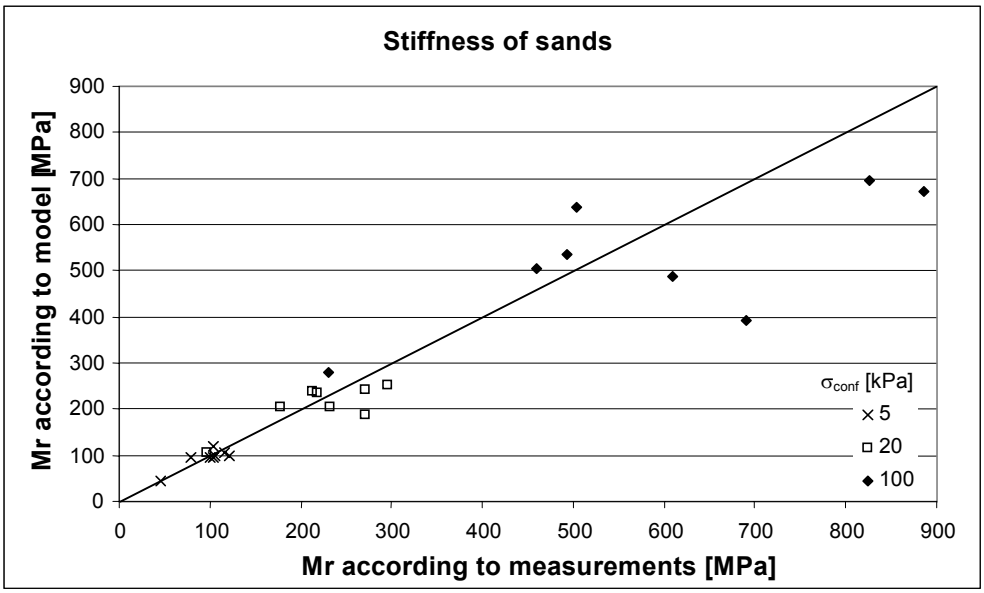

Figure 3. Comparison between modelled stiffness and stiffness on basis of material research.

3.4 Permanent Deformation

It was not possible to develop a model that described the permanent deformation behaviour of sands as a function of grading, grain shape and degree of compaction. For this reason, fixed values for the parameters in equation 3 were assumed. The universal parameters are such that the explained permanent strain development in sands is often conservative. The universal parameters selected for the development of axial strain (first principal strain) were:

$$a1 = -6.52\ [\%],\ a2 = 6\ [-],\ b1 = 0.6\ [-],\ b2 = 5\ [-]$$

Adopting these universal parameters the difference in permanent deformation behaviour between various sands is explained by the values of c and Φ. In strong sands a certain state of absolute stress will lead to low stress ratios, $\sigma_1/\sigma_{1,f}$, and thus to a moderate development of permanent strain. The stress ratio in a weak sand at the same state of absolute stress will be much higher, resulting in much stronger permanent strain development. It was assumed that the permanent radial strain (second and third principal strain) was equal to the axial strain. Of course axial strain is negative while radial strain is positive.

The levels of permanent strain in sands are dependent on the number of applied cycles, the applied stresses, and the strength of the sand. For these reasons it is not easy to provide an insight into the accuracy obtained by using the proposed universal parameters. However, Figure 4 gives some insight into the accuracy of the universal parameters. The figure gives the permanent axial strain after 1 million cycles at a 12 kPa confining pressure as a function of the stress ratio. The continuous line refers to the behaviour obtained on the basis of the proposed universal parameters. The various indicated points follow from measurements on individual sands (Niekerk van, 2002). The legend in the figure gives the abbreviation of the source of the sand followed by the degree of compaction valid for the indicated points. The stress ratio, $\sigma_1/\sigma_{1,f}$, (horizontal axis) increases with the first principal stress but decreases with the strength of the sand.

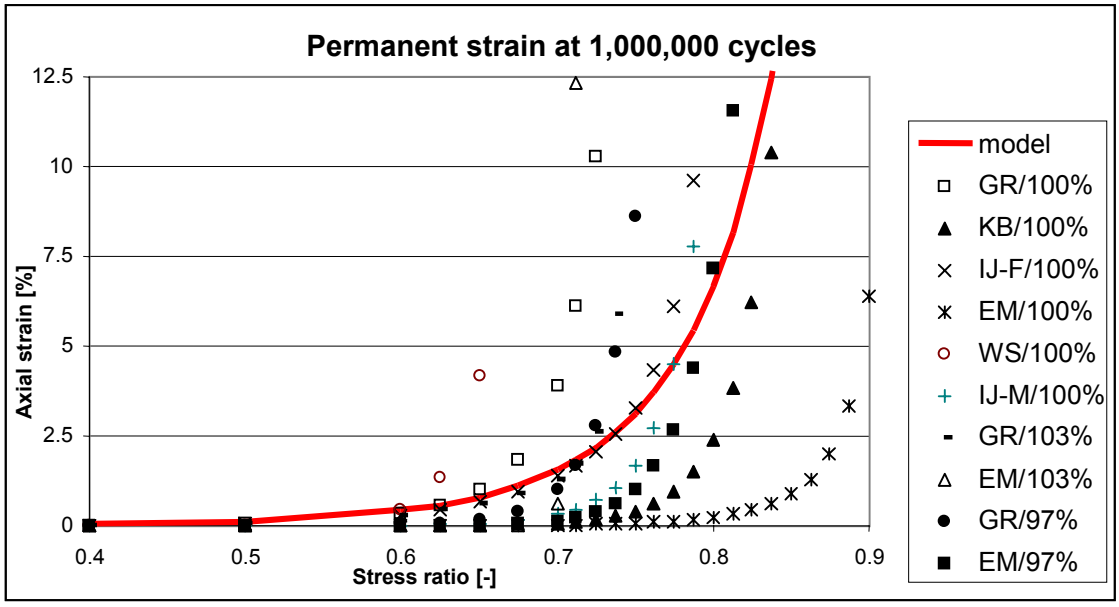

Figure 4. Permanent strain development as explained by the universal model parameters and the model parameters as determined for individual sands.

4. THE DATA SET

For structures with only a sand sub-base 180 calculations were made with the FEM that was developed in the PhD program that ran from 1990 to 1997. This FEM based mechanistic method of explaining cbp behaviour is beyond the scope of this paper and extensively discussed elsewhere (Huurman, 1996a; 1996b; 1997). The outcome of the 180 calculations forms a data set. Each calculation considered the rutting behaviour of a distinct cbp, loaded by a distinct traffic load. The data set will be the basis of the new Dutch design method and its ranges are hereafter briefly discussed.

4.1 Block Layer Behaviour

In the FEM that is the basis of the mechanistic method the block layer is represented by means of blocks interconnected by shear and normal springs. These springs represent the joints between the blocks. The model is fitted with both joint normal and joint shear springs.

In order to reduce the number of calculations needed to come to a design method, all calculations were conducted assuming 80 mm thick standard rectangular blocks with horizontal dimensions of 211 mm x 105 mm (that leads to 44 blocks per m^2 assuming a joint width of 2.5 mm). The effects of other types of block on the structural behaviour of a cbp are incorporated on the basis of a mechanical analysis of the properties of the cb layer.

On the basis of this analysis the mechanical properties of any cb layer of rectangular blocks can be transformed into equivalent joint properties for a cb layer with 80 mm standard blocks. This analysis resulted in the following equation for equivalent joint normal stiffness (a similar equation is derived for the equivalent joint shear stiffness).

$$k_{n,eq} = \frac{44}{n} \frac{(1-\upsilon)E}{(1+\upsilon)(1-2\upsilon)} \frac{h}{2w} \left(\frac{h}{80} \right)^2 \qquad (6)$$

where

$k_{n,eq}$: equivalent joint normal spring stiffness for the structural model per mm modelled joint length [N/mm^2]
n: number of blocks per square meter [-]
ν: Poisson's ratio of joint material [-]
E: Young's modulus of joint material [MPa]
h: thickness of the block [mm]
w: joint width [mm]

The value of $k_{n,eq}$ varied from 400 to 40,500 N/mm^2. Low values are a result of a large joint width, a small size block, thin blocks, poor joint material or combinations thereof.

4.2 Strength of Sand

In the calculations the value of c varied from 0.52 to 45.5 kPa, and the value of Φ varied from 38.1 to 46.4 degrees.

4.3 Stiffness of Sand

The value of k1 varied from 3.4 to 127.9 MPa, while the value of k2 varied from 0.53 to 1.33 [-]. The value of k2 may increase to more than 1 in those cases when the sand is poorly compacted (see Appendix A).

4.4 Thickness of Sand Sub-Base

The thickness of the sand sub-base varied from 500 to 1500 mm.

4.5 Stiffness of Subgrade

The stiffness of the subgrade varied from 30 MPa (peat) to 120 MPa (sand).

4.6 Axle Load

The axle loads varied from 60 to 140 kN.

4.7 Traffic Wander

Lateral traffic wander was taken into account by assuming a standard distribution of the load position. The standard deviation of this distribution varied from 100 to 300 mm.

5. REGRESSION OF DATA SET

The data set briefly discussed in section 4 contains the outcome of the 180 FEM based mechanistic rutting calculations. The output of each individual calculation gives the transversal rut profile as a function of the number of load repetitions. Each calculation gives the transversal rut profile for N = 100, 250, 400, 550, 700, 850, 1000, 2500 … 4000000. From this output the rut depth in the middle of the wheel path is described by means of equation 7. This equation fits almost perfect through the computational data and thus only acts as an accurate data reducer.

$$RD = a_c \cdot \left(\frac{N}{1000} \right)^{b_c} \quad (7)$$

where
a_c: rut depth after 1000 axle load repetitions [mm]
b_c: rate of increase of rut depth on a log-log scale [-]

In order to come to a design method that incorporates a flexible rut depth standard both the values of a_c and b_c should be explained. For this reason equations are developed and presented in Appendix B.

The equations describe the value of a_c and b_c as a function of:
- block layer properties, i.e. the joint normal stiffness $k_{n,eq}$
- sub-base thickness
- strength of the sub-base sand, i.e. c and Φ
- stiffness of the sub-base sand, i.e. k1 and k2
- stiffness of the subgrade
- pavement loading, i.e. axle load and traffic wander

As discussed in section 3, equations giving c and Φ and also k1 and k2 for sands as a function of grain shape (VVS), grading (Cu and d50) and degree of compaction (qc) have been developed. This implies that all information required for the development of a universal design method for block pavements with a sand sub-base only is available.

A computer program for easy design is under development. The code, as it exists now, is based on the above and briefly discussed hereafter.

6. THE DESIGN METHOD

The design method for Dutch ELement PAVEments, DELPAVE, is hereafter discussed on the basis of an example.

DELPAVE consists of three main parts, i.e. access to the road building materials libraries, access to the traffic related libraries and the design procedure itself. The library structure enables an easy to use design procedure based on selecting items from the various libraries as will be explained herafter.

6.1 Materials

There are four road building materials libraries; the small elements libraries, the base materials library (not discussed here), the sands library and the subgrades library.

In the small elements libraries the properties of various small element layers are stored. Figure 5 gives a screen plot of the small elements libraries form. The plot shows the data as stored for the most widely used concrete block in the Netherlands.

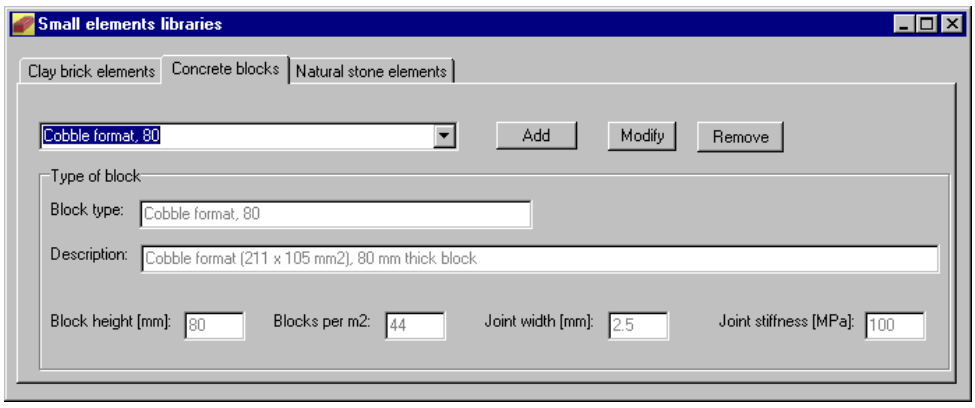

Figure 5. Screen plot of the small elements libraries form.

In the sands library the properties of various sands may be stored. Figure 6 gives a screen plot of the sands library. The plot shows the data as stored for sand dredged from the IJsselmeer fairway.

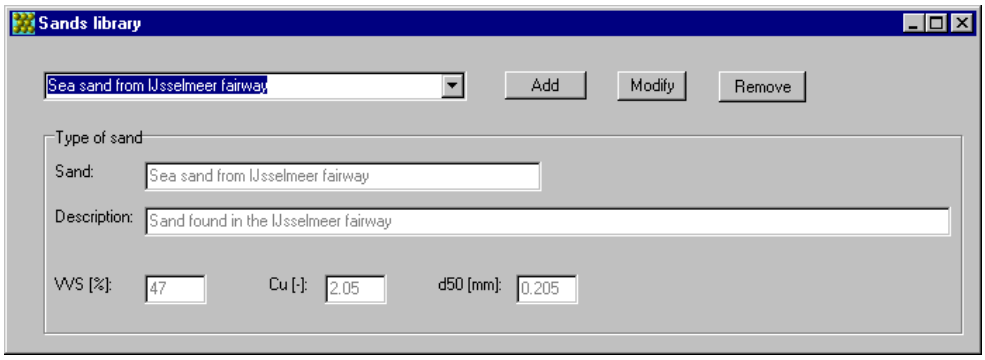

Figure 6. Screen plot of the sands library form.

In the subgrades library the properties of subgrades are stored. Figure 7 gives a screen plot of the subgrades library form. The plot shows the subgrade stiffness as stored for Dutch clay.

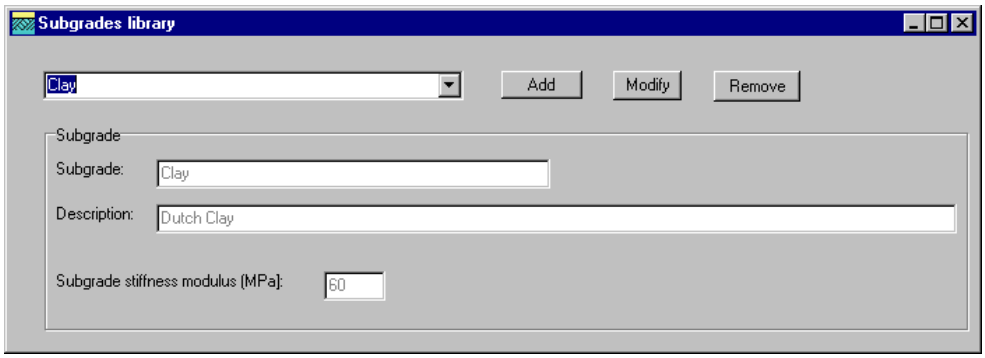

Figure 7. Screen plot of the subgrade library form.

6.2 Traffic

In DELPAVE traffic consists of vehicles. For vehicles the axle load spectrum (left red bars in screen plots) and the contact pressure spectrum (right blue bars in screen plots) are stored in the vehicles library.

Figure 8 gives a screen plot of the vehicles library form. The plot shows the data for a #12 vehicle in the GR03-EUR13 vehicle classification table, i.e. a bus or coach.

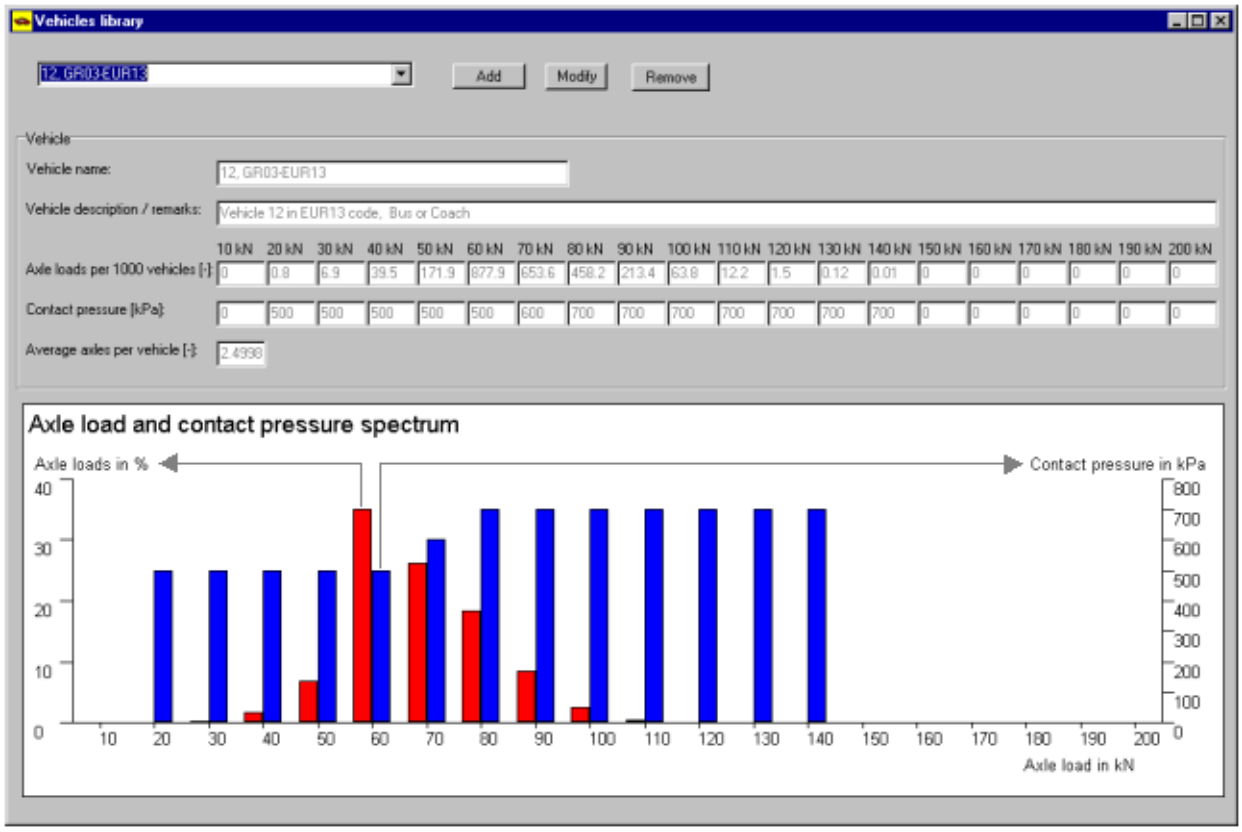

Figure 8. Screen plot of the DELPAVE vehicles library form.

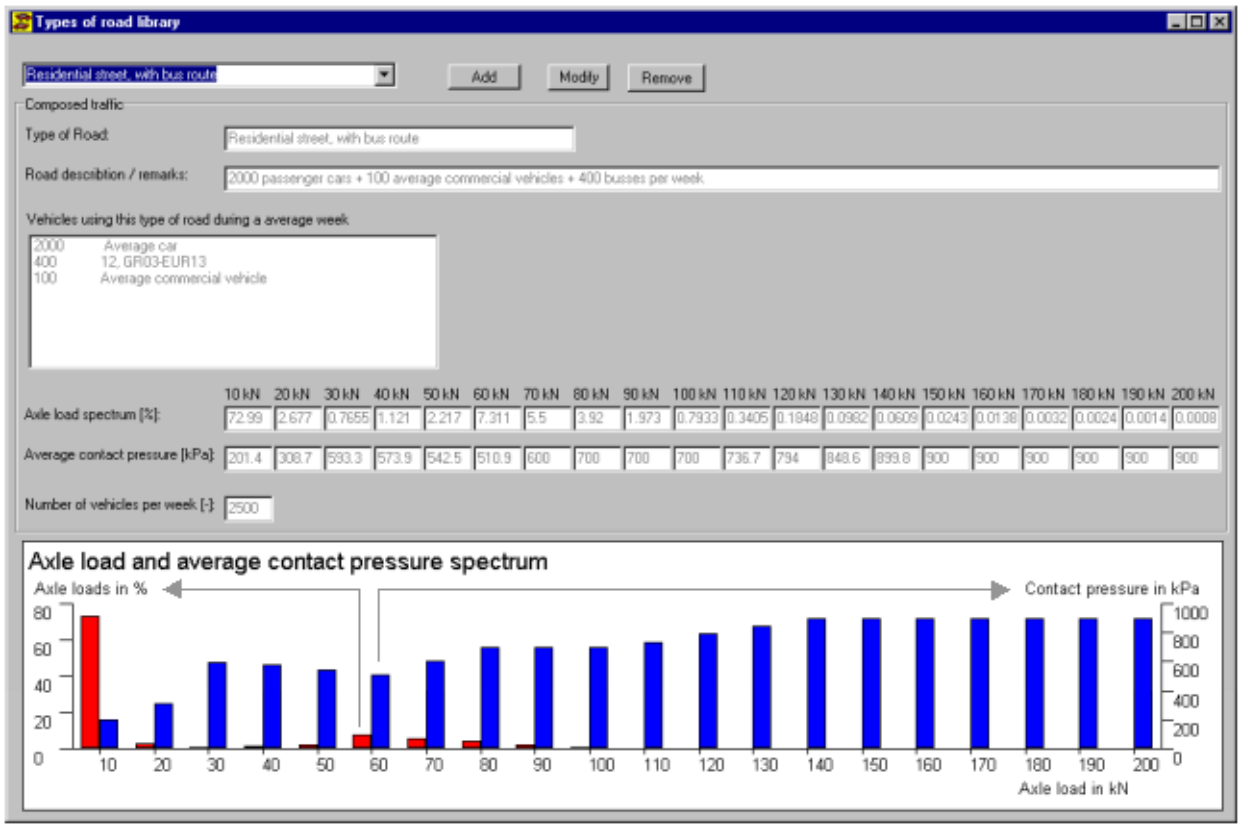

Figure 9. Screen plot of the DELPAVE types of road library form.

6.3 Design

Designing a concrete block pavement with DELPAVE for a large part comes down to selecting items from the various libraries with respect to the type of road, the small element layer, the base layer, the sub-base layer and the subgrade. In some cases additional, design specific, information is required.

Figure 10 shows a screen plot of the design form. The plot shows the input screen with respect to traffic. In this particular input screen three types of additional, design specific, information are required.

Traffic multiplier [-]: This is a multiplier with which the number of vehicle passages, as stored in the types of road library, are multiplied. This multiplier allows for design specific adjustments to the stored traffic load.

Traffic growth [%/yr]: The expected traffic growth per year for the pavement being designed.

Std. dev. of wander [mm]: In DELPAVE traffic wander is reflected by means of the standard distribution of lateral wheel load positions. The standard deviation of this distribution depends on the lane width and as such is design specific input.

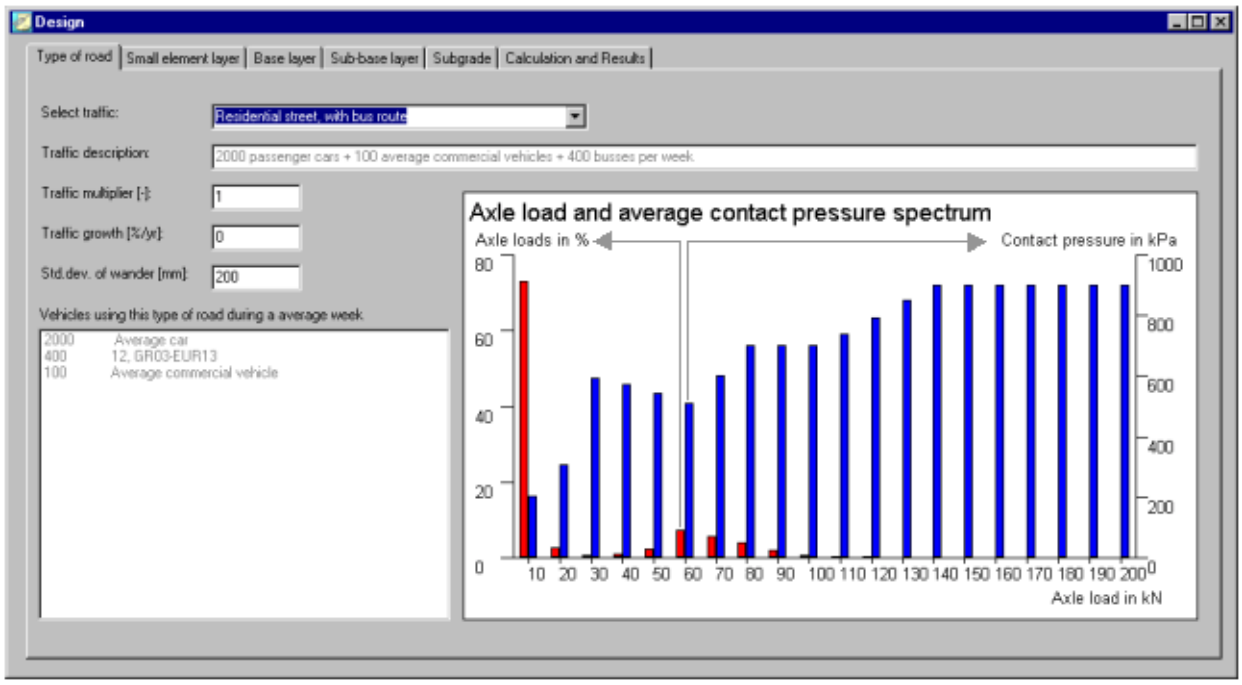

Figure 10. Screen plot of the "Type of road" tab on the Design form.

Figure 10 shows various tabs on the design form. The "Small element layer" tab allows the selection of a small element layer from the element libraries. No additional design specific information is required here.

The "Base layer" tab is not an issue in this paper.

A plot of the "Sub-base layer" tab is given in figure 11. Here design specific additional information about compaction is required. With respect to rutting the degree of compaction of the upper part of the sub-base is of main interest. Compaction of sand in practice is heavily dependent on the subgrade. Compacting thin sand sub-bases on poor subgrades is not possible due to a lack of support of the sub-base layer. For this reason DELPAVE asks for the degree of compaction that is

achieved under ideal circumstances, i.e. "DOC at 5 m thickness [%]:". The degree of compaction that is inputted here is adjusted during the design calculation as will be discussed later. In figure 11 it is shown that the DOC at 5 m thickness [%] is set to be 101.5 % which is a realistic value for high quality compaction in Dutch element paving practice.

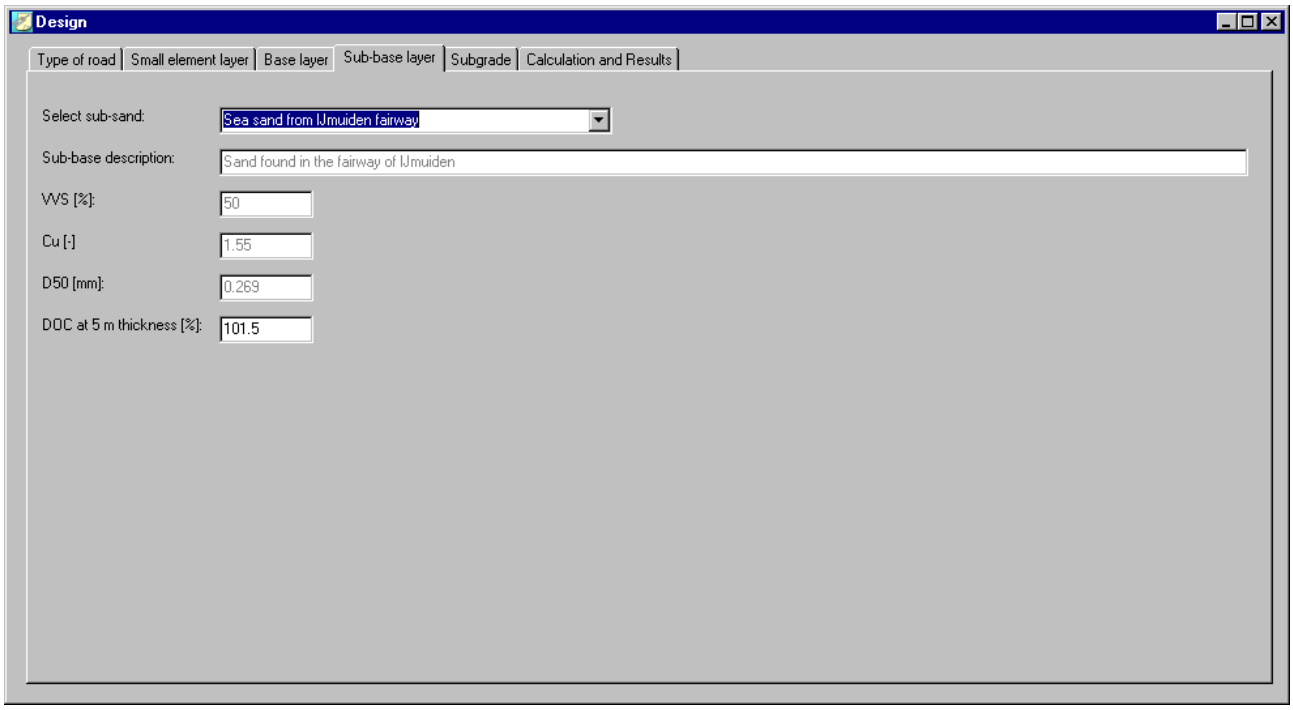

Figure 11. Screen plot of the "Sub-base layer" tab on the Design form.

The "Subgrade" tab allows for the selection of a type of subgrade and is not further discussed here.

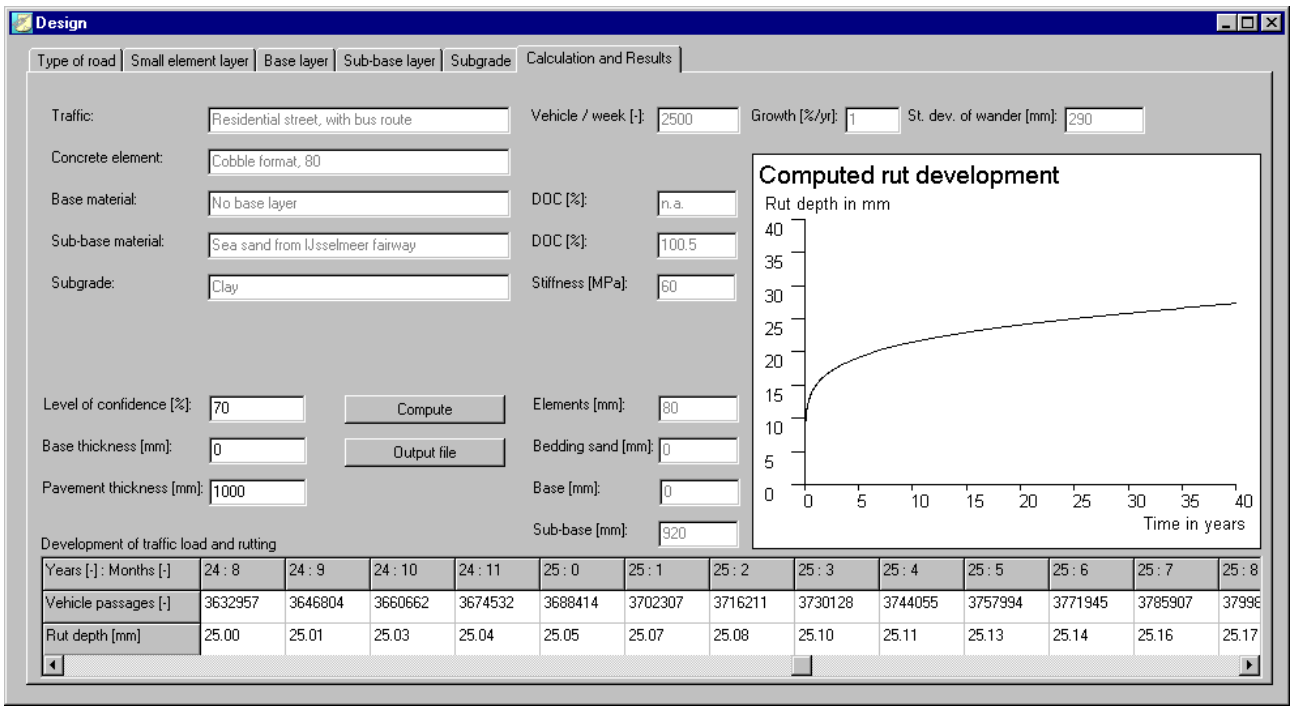

Figure 12. Screen plot of the "Calculation and Results" tab on the Design form.

The last tab on the Design form is the "Calculation and Results" tab. A screen plot of this tab is given in Figure 12. In this particular input screen three types of additional design specific information are required.

Level of confidence [%]: As explained earlier the design method is based on equations that follow from regression on output of a mechanistic FEM based approach of rutting. These equations fit the real computations up to a certain level of accuracy. Explanation of the level of confidence is beyond the scope of this paper, but it is related to the quality of fit of the equations that form the basis of the design method. The value of 70% is considered to be a normal value for concrete block pavement design in the Netherlands.

Base thickness [mm]: Beyond the scope of this paper and thus set to 0 mm.

Pavement thickness [mm]: This value reflects the total thickness of the pavement structure.

In Figure 12 it is shown that the "Calculation and Results" tab gives a brief summary of the input in the upper part of the screen. After pressing the "Compute" button additional information is presented. As shown the DOC of the sub-base is explicitly plotted. The value that is plotted in the screen equals 100.5% which does not equal the earlier discussed input of 101.5%. This is a result of the relation between the DOC under ideal circumstances, the subgrade stiffness and the sub-base thickness. The relation that is part of DELPAVE is presented in Figure 13.

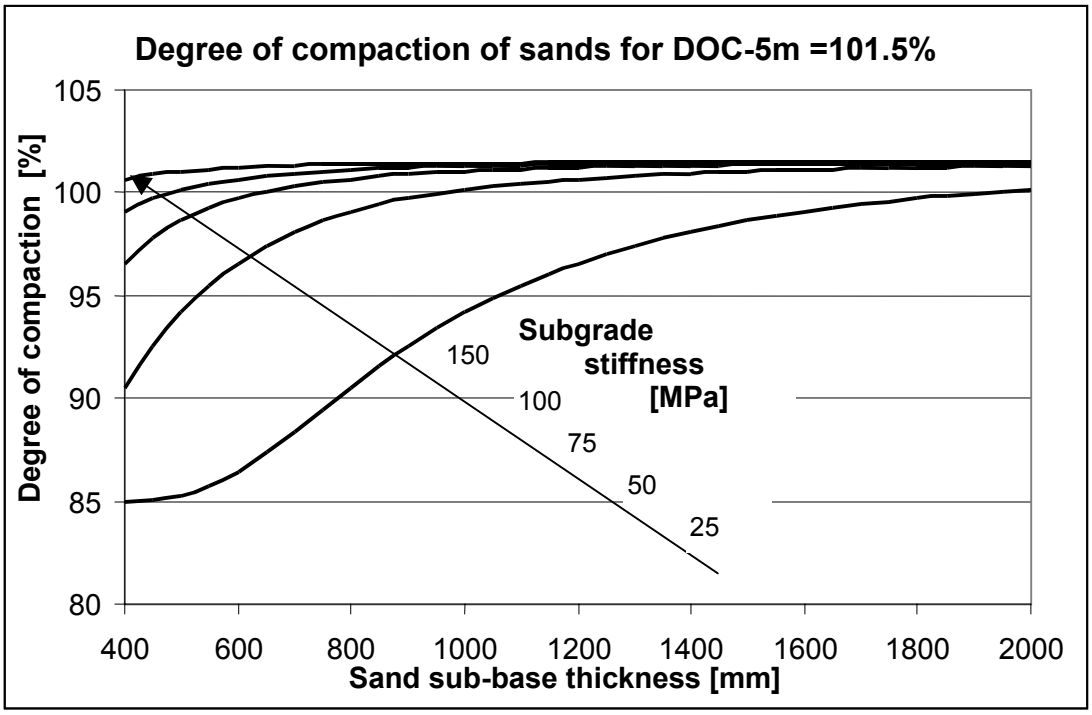

Figure 13. Relation between sub-base DOC, subgrade stiffness and sub-base thickness.

Output is given by means of a plot that gives rutting as a function of time. At the bottom of the screen the output is given with a step size of one month numerically. Since DELPAVE is designed to be a universal design tool it does not consider a specific design rut depth. Figure 12 shows that the input discussed earlier results in 25 mm rut depth after 24:8 [yr:month].

7. DESIGN EXAMPLES

Insight into DELPAVE output is given in this section. The following standard situation is considered.

Type of road:	Residential street with bus route
Traffic multiplier [-]:	1
Traffic growth [%/yr]:	1
St.dev. of traffic wander [mm]:	290
Concrete blocks:	80 mm thick standard blocks (211 mm x 105 mm)
Sub-base sand:	IJsselmeer fairway sand (IJ-M)
DOC at 5 m thickness [%]:	101.5 %
Subgrade:	Dutch clay (60 MPa)
Level of confidence [%]:	70
Pavement thickness [mm]:	1000
Design rut depth [mm]:	25

The above standard situation equals the input discussed in the previous section and results in a 24:8 [yr:month] 25 mm design life.

In Table 3 the effects of the individual parameters on the pavement's design life is shown. The calculations that refer to the above standard situation are marked. By comparison of the various effects it is shown that the main factors effecting the pavement design life are type of sand, sand compaction, design rut depth and subgrade stiffness.

Table 3. Effects of individual parameters on pavement design life.

Concrete block pavement design life			
Concrete block thickness [mm]	70	80	100
25 mm design life [years:months]	18:4	24:8	40:0 / 24.98 mm
ratio [-]	0.74	1	>1.62
Type of concrete block	Thick format	Cobble format	Waal format
25 mm design life [years:months]	18:2	24:8	25:1
ratio [-]	0.74	1	1.02
Type of sand (see Table 1)	IJ-F	IJ-M	GR
25 mm design life [years:months]	2:2	24:8	40:0 / 21.24 mm
ratio [-]	0.09	1	>>>1.62
DOC at 5 m thickness [%]	95	98	101.5
25 mm design life [years:months]	4:9	10:8	24:8
ratio [-]	0.19	0.43	1
Total thickness of structure [mm]	700	1000	1500
25 mm design life [years:months]	11:1	24:8	29:3
ratio [-]	0.45	1	1.19
St.dev. of traffic wander [mm]	200	290	350
25 mm design life [years:months]	18:2	24:8	28:8
ratio [-]	0.74	1	1.16
Subgrade stiffness [MPa]	25	60	150
25 mm design life [years:months]	2:4	24:8	30:10
ratio [-]	0.09	1	1.22
Design rut depth	15	20	25
Design life [years:months]	1:1	6:6	24:8
ratio [-]	0.04	0.26	1
Traffic growth [%]	4	1	0
25 mm design life [years:months]	18:11	24:8	28:0
ratio [-]	0.77	1	1.14

8. CONCLUSIONS

In this paper the development of a universal design method for cbp with unbound structural layers has been discussed.

The main findings are as follows.

- The mechanical behaviour of sands depends strongly on the DOC and the grading and, to a lesser extent, grain shape.
- The behaviour of cbps can be explained by means of a theoretical mechanistic FEM based analysis that considers the cbp structure, the mechanical behaviour of the materials in this structure and the applied traffic loading.
- A universal design method for cbps was developed on the basis of regression on both the mechanical properties of sands determined by triaxial testing and the results of FEM based analysis.
- The use of the universal design method for cbps with a sand sub-base only has been described in this paper.
- Given a certain traffic loading the design life of cbps with a sand sub-base only mainly depends on the subgrade stiffness, the type of sub-base sand, the degree of sub-base compaction and the allowable rut depth.
- The properties of the block layer have a distinct, but secondary, influence on the design life.

9. FUTURE WORK

The method has been completed and is also capable in handling cbps with an unbound base layer and a sand sub-base. The complete method will be published in the near future.

The work presented here is largely based on equations developed during regression analysis. It is the view of the authors that further in-depth insight into cbp structural behaviour asks for theoretical descriptions of the mechanical behaviour of unbound materials in the substructure. Development of such models, based on micro mechanical considerations, will probably take place in the years to come.

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Appendix A

Determination of Parameters Used to Describe the Resilient Behaviour of Sand

k1 is determined using the following relationship.

$$k1 = k11 \cdot \left(\frac{d50}{1mm}\right)^{k12} \cdot \left(\frac{VVS}{100\%}\right) \cdot qc^{k13} \qquad \text{(A1)}$$

where
k11: model parameter = 24.616 [MPa]
k12: model parameter = -0.645 [-]
k13: model parameter = 4.01 [-]

k2 is is determined using the following relationship.

$$k2 = \frac{\ln\left(\frac{Mr_{1000}}{k1}\right)}{\ln(1000)} \quad \text{with} \quad Mr_{1000} = k21 + k22 \cdot \left(\frac{VVS}{100\%}\right)^{k23} Cu \qquad \text{(A2)}$$

where
Mr_{1000}: Mr at 1000 kPa confining pressure combined with small material loading [MPa]
k21: model parameter = 1023.25 [MPa]
k22: model parameter = 30.22 [MPa]
k23: model parameter = -8.264 [-]

k3 is determined using the following relationship.

$$k3 = k31 \cdot \left(\frac{1}{Cu}\right)^{k32} \qquad \text{(A3)}$$

where
k31: model parameter = 2.56 [-]
k32: model parameter = 0.5511 [-]

k4 is determined using the following relationship.

$$k4 = k41 \cdot \left(\frac{d50}{1mm \cdot Cu}\right) \qquad \text{(A4)}$$

where
k41: model parameter = 46.87 [-]

It is noted that the value of parameter k13 was assumed to be 4.01. This parameter describes the dependency of Mr at low confinement stress on qc, the quality of compaction. The value of 4.01 is based on analysis of base material behaviour (not discussed here and not yet published). The value of k13 cannot be verified since the available data does not include variation of the degree of compaction. It was also assumed that the stiffness at high confinement, Mr_{1000}, did not depend on the degree of compaction. This assumption is also based on base material behaviour.
As a result of these assumptions, the value of k1 increases with an increase in compaction level while the value of k2 decreases. At a low level of compaction a small k1-value is combined with a high k2-value. Due to these assumptions the value of k2 may be larger than 1 at low degrees of compaction.

Appendix B

Determination of Values of a_c and b_c

The values of a_c are determined using the following equations.

$$a_c = aa \cdot Mr_a \cdot S_a \cdot ET_a \cdot LKW_a \quad \text{(B1)}$$

where

aa: model parameter = 1.803 [mm]
Mr_a: factor describing the effects of sand stiffness [-]
S_a: factor describing the effects of sand strength [-]
ET_a: factor describing the effects of sub-base thickness and subgrade stiffness [-]
LKW_a: factor describing the effects of the axle load, the block layer and traffic wander [-]

Mr_a, effect of sand stiffness.

$$Mr_a = 1 + \left(\frac{ab}{Mr_{repr}} \right)^{ac} \quad \text{with} \quad Mr_{repr} = k1(ad)^{k2} \quad \text{(B2)}$$

where

ab: model parameter = 1.612 [-]
ac: model parameter = 5.0 [-]
ad: model parameter = 0.611 [-]

S_a, effect of sand strength.

$$S_a = \left(\frac{ae}{S_{repr}} \right)^{af} \quad \text{with} \quad S_{repr} = c + ag \cdot \tan(\phi) \quad \text{(B3)}$$

where

ae: model parameter = 4024 [kPa]
af: model parameter = 0.311 [-]
ag: model parameter = 1.58 [kPa]

ET_a, effect of sub-base thickness and subgrade stiffness.

$$ET_a = 1 + ah \cdot \exp\left(-\frac{Es}{100\ MPa} \right) \cdot \exp\left(\left(-\left(\frac{Tsb}{ai} \right)^{aj} \right) \right) \quad \text{(B4)}$$

where

Es: subgrade stiffness [MPa]
Tsb: sub-base thickness [mm]
ah: model parameter = 1.612 [-]
ai: reference sub-base thickness = 488.3 [mm]
aj: model parameter = 5.0 [-]

LKW_a, effect of axle load, joint stiffness (i.e. block layer behaviour) and lateral wander.

$$LKW_a = \left(\frac{L}{100\ kN} \right)^{ak} \cdot \left(\frac{5500\ N/mm^2}{k_{n,eq}} \right)^{ak} \cdot \left(\frac{100\ mm}{LW} \right)^{am} \quad \text{(B5)}$$

where

L: axle load [kN]
$k_{n,eq}$: equivalent joint normal spring stiffness [N/mm^2]

LW: standard deviation of lateral wander
ak: model parameter = 0.853 [-]
al: model parameter = 0.080 [-]
am: model parameter = 0.179 [-]

The values of b_c as found in the data set are described as follows.

$$b_c = \left(Mr_b + \cdot S_b\right) \cdot LKW_b \tag{B6}$$

where
Mr_b: factor describing the effects of sand stiffness [-]
S_b: factor describing the effects of sand strength [-]
LKW_b: factor describing the effects of the axle load, the block layer and traffic wander [-]

Mr_b, effect of sand stiffness.

$$Mr_b = \left(\frac{ba}{Mr_{repr}}\right)^{bb} \quad \text{with} \quad Mr_{repr} = k1(bc)^{k2} \tag{B7}$$

where
ba: model parameter = 0.0268 [-]
bb: model parameter = 5.0 [-]
bc: model parameter = 0.0246 [-]

S_b, effect of sand strength.

$$S_b = bd \cdot \exp\left(-\left(\left(\frac{c}{be}\right)^{bf}\right)\right) \tag{B8}$$

where
bd: model parameter = 0.204 [-]
be: model parameter = 51.13 [kPa]
bf: model parameter = 0.9687 [kPa]

LKW_b, effect of axle load, joint stiffness (i.e. block layer behaviour) and wander.

$$LKW_b = \left(\frac{L}{100\ kN}\right)^{ag} \cdot \left(\frac{5500\ \ N/mm^2}{k_{n,eq}}\right)^{ah} \cdot \left(\frac{100\ mm}{LW}\right)^{ai} \tag{B9}$$

where
L: axle load [kN]
$k_{n,eq}$: equivalent joint normal spring stiffness [N/mm^2]
LW: standard deviation of lateral wander
ag: model parameter = 0.179 [-]
ah: model parameter = 0.055 [-]
ai: model parameter = -0.0396 [-]

THE UPGRADED DUTCH DESIGN METHOD FOR CONCRETE BLOCK ROAD PAVEMENTS

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CURRICULLUM VIIAE
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Rien has a 15-year history in developing mechanistic models and tools for CBP design purposes. In 1989 he graduated as an engineer at the Delft University of Technology on the development of a full 3D-FEM for CBPs. After completion of military cervices he returned to Delft to further work on modelling CBP behaviour as a PhD candidate. He developed a FEM-code that allowed the calculation of rutting -in CBPs with unbound materials only- as a function of CBP structural design, traffic loading and material properties obtained from triaxial testing. Material stiffness, strength and permanent deformation behaviour proved to be important and all showed stress dependency. By means of repeated vehicle simulation he also investigated the development of longitudinal unevenness due to traffic-CBP interaction. In 1997 he completed this work.
Hereafter he left Delft for a 5-year period. He returned to the university in 2001 as an assistant Professor. It was soon hereafter that he started his work on the development of a universal design method for small element pavements. In his paper and presentation he gives us more information about this new method that is available as a Windows application in the Netherlands.