RESILIENT BEHAVIOUR OF CONCRETE BLOCK PAVEMENTS AND THE GRANULAR MATERIALS USED IN THE SUBSTRUCTURE

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Summary

At the Delft University of Technology a research program is performed to gain more knowledge about the development of permanent deformation (rutting) in Concrete Block Pavements (CBP), caused by traffic loadings. For this research a good insight into the stresses introduced in the substructure of a CBP is needed.
In this paper the resilient properties of eight Dutch sands and four Dutch unbound base course materials are discussed. It is shown that the most commonly used material model, the Mr-Θ model, results in stresses that indicate failure of CBP with only a sand sub-base, which are very commonly applied in the Netherlands.
A more advanced model to describe the resilient behaviour of the sands as a function of the confining stress and the failure-ratio (stress/strength) is discussed. It is shown that this model describes the behaviour of the sands much better than the Mr-Θ model does. The stresses computed on the basis of this more advanced model do not indicate failure of CBP without a base and thus are in better agreement with reality.
Some additional computations which refer to CBP with an unbound base and a sand sub-base are discussed. It is shown that the positive effects of a base layer on the resilient deformations of a CBP are rather limited. The effects of a base course layer on the stresses in the substructure are much larger. It is shown that a base layer reduces the failure-ratios in the most heavily loaded area of the substructure enormously.

1. Introduction

The goal of the research is to explain and understand the development of rutting in Concrete Block Pavements (CBP). For this reason insight into the development of permanent strain in the substructure of a CBP is needed. The development of these permanent strains heavily depends on the stresses in the substructure due to a wheel load, as is discussed in the paper "Rut development in Concrete Block Pavements due to permanent strain in the substructure" [1].
This paper deals with the resilient behaviour of CBP. The stress-dependent resilient properties of eight sands and four unbound base course materials are first discussed. It is shown that these stress-dependent resilient properties have to be taken into account in order to come to computations that result in realistic substructure stresses.
This paper ends with a discussion on the calculation results for three CBP. Two of these CBP have a base and a sand sub-base, while one of them only has a sand sub-base. In [1] it is discussed how these results can be used to explain the development of permanent strain in the substructure of these CBP.

2. Material Behaviour

2.1 *General*

The material properties that are needed to predict the permanent strain development in the substructure of a CBP consider:
- strength,
- resilient behaviour,
- permanent strain development.

For this research eight sands (seven natural sands and one crusher sand) and four unbound base materials were tested in the Laboratory for Road and Railroad Research of the Delft University of Technology. Except for the crusher sand al materials are named after the road from which they were taken, for instance "Zaanweg sand".
In this paper the strength and the resilient behaviour of the materials are discussed, emphasis is laid on the behaviour of the sands. The behaviour of the base course materials is only roughly discussed.

2.2 *Test set-up*

The earlier mentioned material properties are all determined using a triaxial test set-up [2, 3, 4], see figure 1. For testing a sand a cylindrical sample with a height of 200 mm and a diameter of 101.6 mm is prepared. This sand sample is inclosed in a membrane and placed in a pressure cell. The sand sample is connected with the atmosphere while the pressure in the cell can be varied so that the sample can be subjected to an adjustable confining pressure σ_{conf}.
After σ_{conf} is applied the actuator is used to subject the sample to an additional stress in the vertical direction.
The triaxial test set-up for testing base course materials is much larger than the set-up used for testing sands and requires samples with a height of 800 mm and a diameter of 400 mm. In this larger test set-up σ_{conf} is applied by lowering the pressure in the

sample (pore pressure). The atmospheric pressure now confines the sample.

2.3 *Strength*

The strength of a granular material, expressed in the first principal stress at failure $\sigma_{1,f}$, strongly depends on σ_{conf}. The relation between $\sigma_{1,f}$ and σ_{conf} can be explained using the well known Mohr-Coulomb criterion. In this research program the cohesion C and angle of internal friction ϕ are determined on the basis of three static failure triaxial tests. In these tests, performed at three levels of σ_{conf}, the actuator imposes the sample to a slowly increasing vertical strain in a displacement controlled mode. The actuator force and displacement are recorded, the maximum force is used to determine the first principal stress at failure $\sigma_{1,f}$.

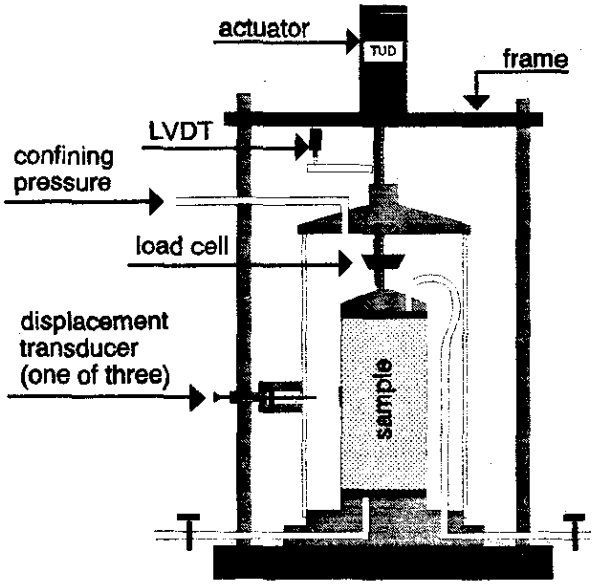


Fig. 1 The triaxial test set-up, sample size 200 x 101.6 mm².

For illustration in figure 2 the results obtained for the one natural sand, the Zaanweg sub-base sand, are plotted. As shown by this plot a very good fit between the data and the Mohr-Coulomb criterion was obtained.

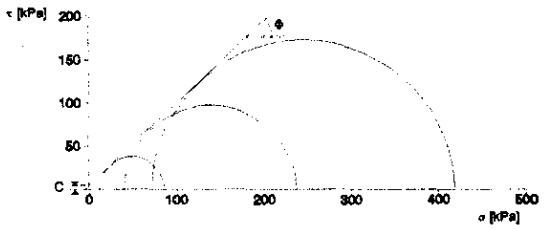


Fig. 2 Failure test results obtained for the Zaanweg sub-base sand.

Sand	C [kPa]	ϕ[°]	Base material	C [kPa]	ϕ[°]
Allanstraat	6.3	41.8	Allanstraat	45.9	41.6
Weiver	6.8	43.0	M. Havelaarweg	73.7	43.7
Baarsweg	7.2	42.8	C. Bruynweg	58.8	49.6
M. Havelaarweg	7.5	42.9	Pascalweg	68.7	51.0
Zaanweg	4.1	43.9			
C. Bruynweg	5.6	48.2			
Pascalweg	8.0	39.7			
Crusher	8.7	50.2			

Table 1

Values for C and ϕ found for the tested materials.

Similar results were obtained for the other sands, see table 1. As shown by table 1

base course materials show a far larger Cohesion than the sands. Considering the angle of internal friction no major difference is found between base course materials and sands.

2.4 *Resilient Behaviour*

For the determination of the resilient behaviour of a granular material a cyclic load triaxial test is performed. In these tests the stresses applied to the sample are more complex. Again, first of all σ_{conf} is applied. Hereafter a very small static stress σ_{stat} is applied by the actuator. The last stress applied by the actuator is a halfsine-shaped cyclic stress σ_{cycl}.
During testing σ_{cycl} as well as the sample deformations as a result of this stress are continuously recorded. The measured resilient sample deformations under the cyclic load are transferred to the resilient radial strain, ϵ_{rad}, and the resilient axial strain, ϵ_{ax}. The resilient modulus, Mr, and the Poisson's ratio, ν, can now be determined using equation 1 and equation 2 respectively.

$$Mr = \frac{\sigma_{cycl}}{\epsilon_{ax}}, \qquad \nu = -\frac{\epsilon_{rad}}{\epsilon_{ax}} \qquad (1), (2)$$

In the described triaxial test σ_2 and σ_3 equal σ_{conf}. The first principal stress σ_1 is equal to the sum of the following vertical stresses:

$$\sigma_1 = \sigma_{conf} + \sigma_{stat} + \sigma_{dw} + \sigma_{cycl} \qquad (3)$$

In this equation σ_{dw} represents the stresses as a result of dead weight.
Both Mr and ν appear to be stress dependent. Various models are at hand to describe the stress dependency of Mr. Most of these resilient behaviour models describe the measured stiffness Mr as a function of the sum of the principal stresses Θ:

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 \qquad (4)$$

The most common model describing the stress dependency of Mr is the Mr-Θ model, introduced by Brown and Pell [5].

$$Mr = k1\left(\frac{\theta}{\theta_0}\right)^{k2} \qquad (5)$$

where:
k1: model parameter [MPa]
k2: model parameter [-]
Θ_0: reference stress = 1 kPa [kPa]

In figure 3 the Mr-Θ model is presented together with the measured data obtained for the Zaanweg sub-base sand.
Similar results are obtained for the other sands and the base course materials, see table 2.

As shown by figure 3 and table 2, the Mr-Θ model describes the general trend in the data quite well. However, by distinguishing the various levels of σ_{conf} at which Mr values were determined for the sands, one comes to a completely different conclusion.

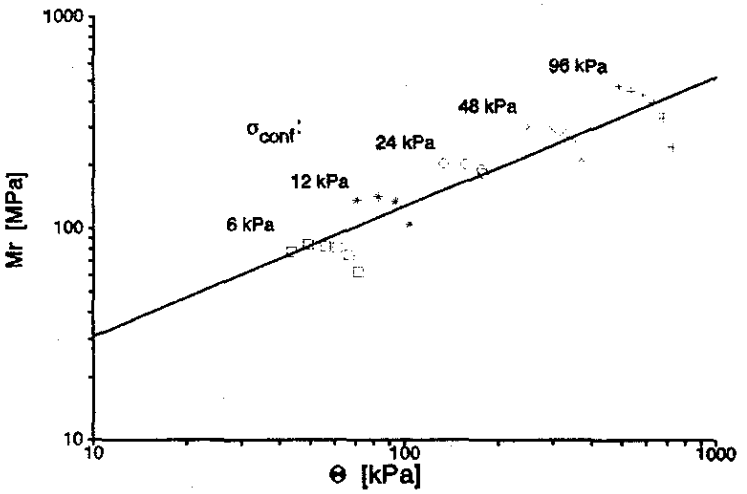


Fig. 3 The Mr-Θ model combined with the Mr measuring data obtained for the Zaanweg subbase sand.

Per σ_{conf} level, σ_2 and σ_3 are constant. This implies that the increase of Θ, at a constant level of σ_{conf}, completely depends on an increase of σ_1. For a constant level of σ_{conf} the data in figure 3 clearly shows a decreasing Mr with an increasing Θ. An increase of σ_1 thus leads to a decrease of Mr. The same data however shows that an increase in σ_{conf}, i.e. σ_2 and σ_3, leads to an increase of Mr.

For the other seven sands similar results are obtained.

Sand	k1 [MPa]	k2 [-]	r^2	Base material	k1 [MPa]	k2 [-]	r^2
Allanstraat	18.6	0.48	0.81	Allanstraat	32.1	0.43	0.42
Weiver	10.1	0.60	0.82	M. Havelaar	61.5	0.34	0.84
Baarsweg	12.6	0.55	0.84	C. Bruynweg	20.2	0.54	0.79
M. Havelaarweg	6.3	0.72	0.77	Pascalweg	29.0	0.39	0.86
Zaanweg	7.4	0.62	0.85				
C. Bruynweg	5.4	0.69	0.82				
Pascalweg	8.2	0.67	0.79				
Crusher	5.7	0.55	0.88				

Table 2

Model parameters found for the Mr-Θ model.

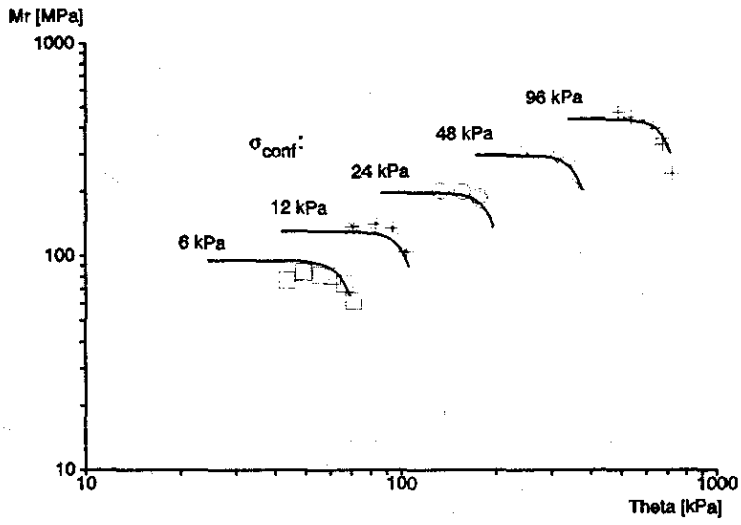
It might be clear that the conventional Mr-Θ model is not able to describe this behaviour since an increase of both σ_{conf} or σ_3 and σ_1 lead to an increase of Θ and, according to the model, thus to an increase of Mr.

Therefore a new model was developed that is capable to describe the effects of both σ_3 and σ_1 on the development of Mr, equations 6 and 7. This new model is applied for the sands. The resilient behaviour of the base materials is described by the Mr-Θ model since these materials did not show a decrease of Mr with increasing σ_1.

$$Mr = k5\left(\frac{\sigma_3}{\sigma_{3_0}}\right)^{k6}\left(1 - k7\left(\frac{\sigma_1}{\sigma_{1,f}}\right)^{k8}\right) \quad (6) \qquad with \quad \sigma_{1,f} = \frac{(1+\sin\phi)\,\sigma_3 + 2c\cos\phi}{1-\sin\phi} \quad (7)$$

where:

$\sigma_{1,f}$:	σ_1 at failure according to Mohr-Coulomb [kPa]
σ_{3_0}:	reference stress = 1 kPa [kPa]
k5:	model parameter [MPa]
k6, k7, k8:	model parameters [-]



The first term in equation 6 describes the increase of Mr with an increase of σ_3. This dependency of Mr on σ_3 was earlier found by others [6]. The second term, describing the decrease of Mr with an increase of σ_1 is added to the Mr-σ_3 model to obtain a model that describes the measured data much better than the Mr-Θ model does, see figure 4.

Fig. 4 The Mr-σ_3-σ_1 model and the measured Mr-values for the Zaanweg sub-base sand.

As shown by table 3 similar results were obtained for the other sands.

Sand	k5 [MPa]	k6 [-]	k7 [-]	k8 [-]	r^2
Allanstraat sand	45.95	0.52	0.36	7.05	0.96
Weiver sand	38.40	0.60	0.89	5.79	0.98
Baarsweg sand	44.96	0.52	0.28	5.83	0.98
Max Havelaarweg sand	38.64	0.68	0.42	4.04	0.94
Zaanweg sand	30.42	0.59	0.47	7.68	0.98
Cor Bruinweg sand	47.75	0.58	0.67	1.51	0.95
Pascalweg sand	34.43	0.69	0.37	4.97	0.97
Crusher sand	19.23	0.54	0.19	8.63	0.99

Table 3: Model parameters found for the Mr-σ_3-σ_1 model on different sands.

As mentioned earlier, the Poisson's ratio, ν, also shows a stress dependency. A simple equation is used to describe this ratio as a function of the $\sigma_1/\sigma_{1,f}$-ratio:

$$\nu = \nu 0 + \nu 1 \left(\frac{\sigma_1}{\sigma_{1,f}} \right)^{\nu 2} \qquad (8)$$

Where:
ν0, ν1, ν2: model parameters [-]

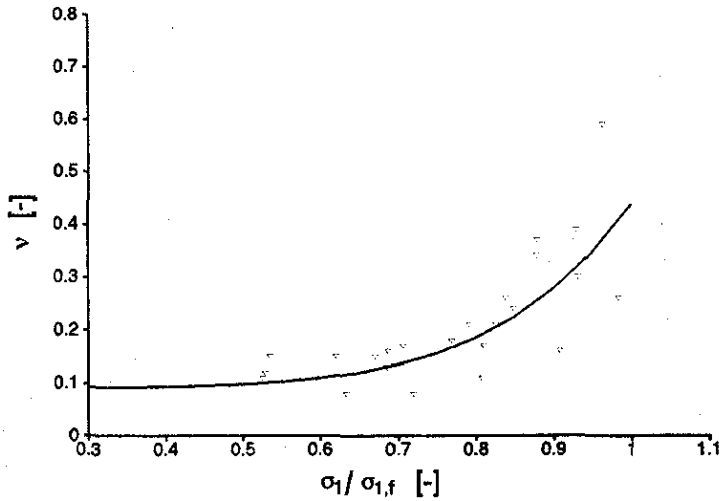


Fig. 5 The stress dependent ν as measured for the Zaanweg sand.

In figure 5 the stress dependent Poisson's ratio and this model are both shown for the Zaanweg sub-base sand. Table 4 shows that similar results are obtained for the other sands.
The results obtained for the sands show that the sands have a ν-value that is smaller than 0.5 at small $\sigma_1/\sigma_{1,f}$. With an increase of the $\sigma_1/\sigma_{1,f}$-ratio the ν shows an increase too.

Sand	ν0 [-]	ν1 [-]	ν2 [-]	r^2
Allanstraat sand	0.12	0.27	6.80	0.87
Weiver sand	0.17	0.58	4.23	0.91
Baarsweg sand	0.15	0.23	4.61	0.89
Max Havelaarweg sand	0.04	0.40	1.48	0.59
Zaanweg sand	0.09	0.35	5.82	0.66
Cor Bruinweg sand	0.13	0.45	3.95	0.30
Pascalweg sand	0.10	0.31	2.39	0.69
Crusher sand	0.11	0.54	1.72	0.97

Table 4: Parameters found for the ν-$\sigma_1/\sigma_{1,f}$ model for sands.

For the base course materials much larger ν-values were measured. In case of a base material the ν at small stress-ratios is about 0.4. With an increase of the stress-ratio,

the ν increases too. As a result, ν-values larger than 0.5 are measured even when the $\sigma_1/\sigma_{1,f}$-ratio is much smaller than 1.
A ν larger than 0.5 can not be applied in the finite element calculations discussed later, therefor the ν of the base course materials is modelled by a constant value of 0.49.

3. Resilient behaviour of Concrete Block Pavements

3.1. *Introduction*

For the analysis of the resilient behaviour of concrete block pavements the stress dependent resilient behaviour of the unbound granular materials, applied in such pavements, clearly has to be taken into account. For this reason an axial symmetric finite element model was developed. This finite element model is extensively discussed elsewhere [7]. In this chapter the most important characteristics of this model are briefly discussed.
Hereafter some calculation results obtained using this model are discussed.

3.2 *The finite element pavement model*

The geometry of the axial symmetric finite element model of the concrete block pavement structure is straight forward, see figure 6. The elements used are simple four node elements with eight degrees of freedom. The size of these elements increases with both the radius and the depth. The smallest elements are found in the centre of the model directly under the loaded concrete blocks and have a height of 25 mm and a width of 32 mm.

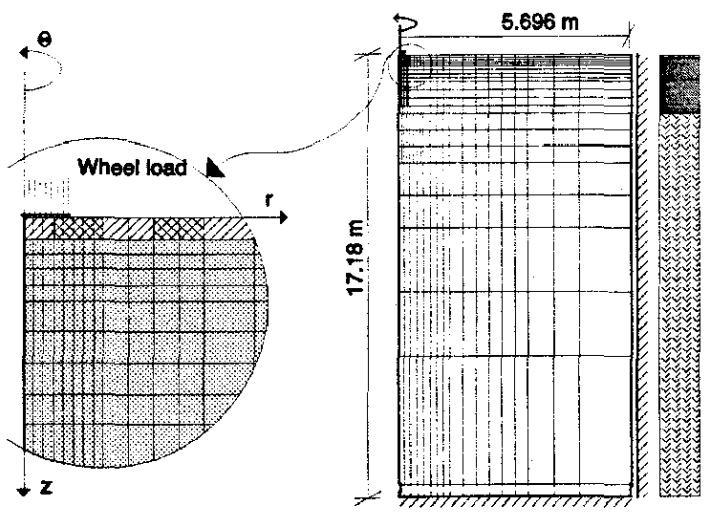


Fig. 6 Basic geometry of the Finite Element Model used for the resilient analysis of concrete block pavements.

In developing this model special attention is given to the representation of the concrete block layer, see figure 7.
The block pattern that is most commonly used in the Netherlands, herringbone bond, does certainly not answer to axial symmetry. Such a pattern really asks for 3-D modelling.
In order to largely overcome the disadvantages of an axial symmetric representation of the block layer, the elements that are used in the representations of the blocks have been taken their ability to build up normal stresses in the Θ-direction, $\sigma_{\theta\theta}=0$. As a result the blocks now have two additional types of stress-free movements, they can translate in the r-direction and rotate around the Θ-axis.
Through this modification the modelled concrete block layer no longer shows self confining properties as does a block layer modelled by normal axial symmetric

elements. The modelled blocks furthermore are able to rotate and thus introduce joint normal forces. The resilient behaviour of the modelled block layer is now similar to a real concrete block layer.

The joints between the blocks are represented by sets of normal and shear springs, figure 7. These springs are linear elastic and active in case of joint compression, in case of joint expansion (tensile normal joint forces) both springs are inactive.
The stiffnesses of the joint springs are based on linear elastic back calculations of FWD deflection bowls measured on the pavements from which the tested sands and base materials were taken.
For the normal joint springs a stiffness of 550 N/mm per mm modelled joint was found. The shear joint springs have a stiffness of 50 N/mm/mm. These joint spring stiffnesses refer to 80 mm thick concrete blocks.

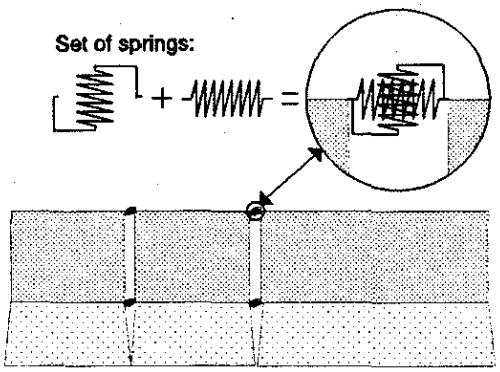


Fig. 7 Representation of the concrete block layer.

3.3 *Effects of Material Modelling*

In this paragraph the effects of the modelling of the resilient behaviour of the sands on the calculated stresses are discussed. Hereto three calculations are performed. These calculations all refer to a CBP with only a 1000 mm sand sub-base placed over the subgrade (E=60 MPa, ν=0.45). The thickness of the blocks is 80 mm (E=40,000 MPa, ν=0.25). The load is a single wheel load, the radius of the contact area is 150 mm.
In the first calculation the sand in the sub-base is modelled by a constant Young's modulus of 100 MPa combined with a Poisson's ratio of 0.35 (very commonly used values for sands in the Netherlands). In the second calculation the Mr-Θ model is applied in combination with a constant ν of 0.35. In the third calculation the Mr-σ_3-σ_1 model is used in combination with a stress dependent ν. The model parameters used in the last two calculations refer to the Zaanweg sub-base sand.

In figure 8 the deflection bowls calculated using the three material models are given. As shown by this figure, only minor differences in the calculated deflection bowls are found.

In order to explain the development of permanent strain in the substructure of a pavement (rutting), the stresses in this substructure are however of interest. As is discussed elsewhere [1], especially the $\sigma_1/\sigma_{1,f}$-ratio is important. If this ratio is smaller than one then the stresses do not lead to failure of the material, when this ratio is however larger than one the stresses in the material will lead to failure.

In figure 9 the calculated $\sigma_1/\sigma_{1,f}$-ratio in the centre of the model is plotted against the depth. As shown by this figure major differences are now found between the three

material models compared here.
The Mr-Θ model leads to $\sigma_1/\sigma_{1,f}$-ratio's of about 4.6, meaning that the calculated σ_1 is 4.6 times larger than the allowable σ_1 calculated on the basis of equation 7. Considering the stresses in the substructure computed on the basis of the Mr-Θ model it is thus concluded that the discussed CBP will fail under 50 kN wheel loads.
The same conclusion is drawn when one considers the results obtained on the basis of the linear approach. In this case the maximum $\sigma_1/\sigma_{1,f}$-ratio in the centre of the model equals about 1.3.
In The Netherlands however most CBP do not have a base. These CBP without a base of course show rutting, failure of such CBP however hardly occurs.

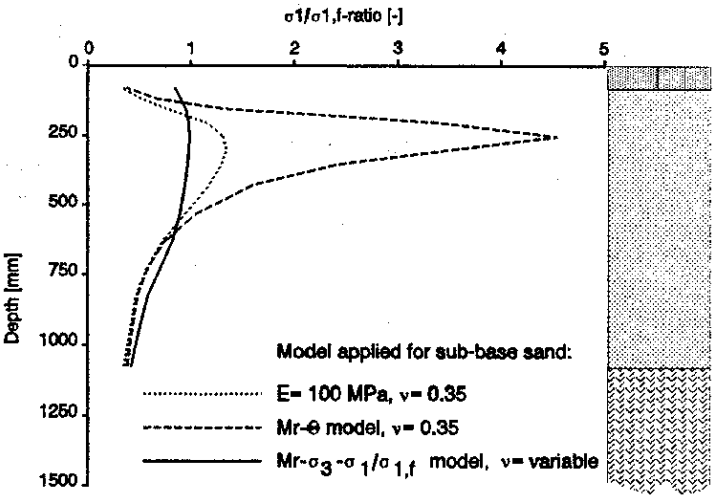


Fig. 8 Calculated $\sigma_1/\sigma_{1,f}$-ratio using the three different material models for the sub-base sand.

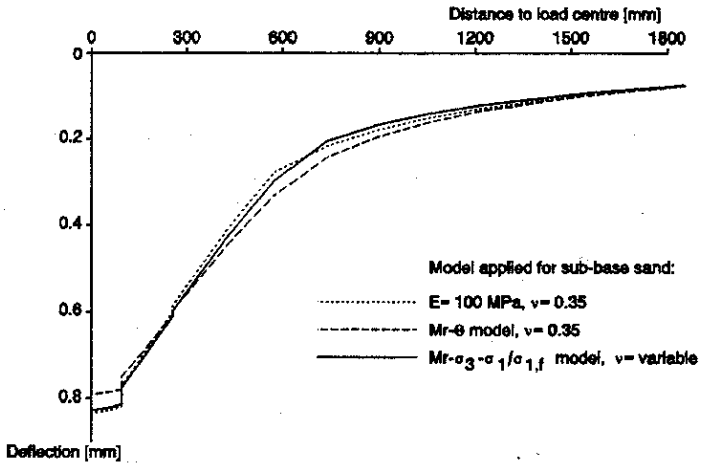


Fig. 9 Deflections calculated using three models for the resilient behaviour of the sub-base sand.

The stresses calculated on the basis of the Mr-Θ model and the linear model thus do not agree with reality. When the Mr-σ_3-σ_1 model is used for the sub-base sand the calculated $\sigma_1/\sigma_{1,f}$-ratios are smaller than one, indicating that the CBP will not fail under a 50 kN wheel load. The calculated $\sigma_1/\sigma_{1,f}$-ratios are however very close to one which implies that permanent strains in the substructure will develop rapidly when the CBP is subjected to 50 kN wheel loads. It might be clear that these conclusions are in agreement with the experience based on the behaviour of Dutch CBP without a base.

3.4 *Effects of a Base Course Layer*

In this paper three pavement analyses are discussed in order to show the effects of a base course layer. In all cases the total pavement structure has a height of 1080 mm. The first layer has a thickness of 80 mm and represents the concrete block layer (E= 40,000 MPa, μ= 0.25). Pavement structure 1 only has a 1000 mm thick sand sub-base

(stress dependent resilient behaviour of the Zaanweg sub-base sand) and thus equals the CBP discussed in the previous paragraph.
The other two pavements have an unbound granular base. These pavements both have a 50 mm thick bedding sand layer (stress dependent resilient behaviour of crusher sand). Below this bedding layer the base layer is situated (stress dependent behaviour of the Pascalweg base course material). In case of pavement structure 2 this base has a thickness of 200 mm, while structure 3 has a 300 mm thick base. These base layers are placed over a Zaanweg sub-base sand layer with a thickness of 750 and 650 mm respectively.

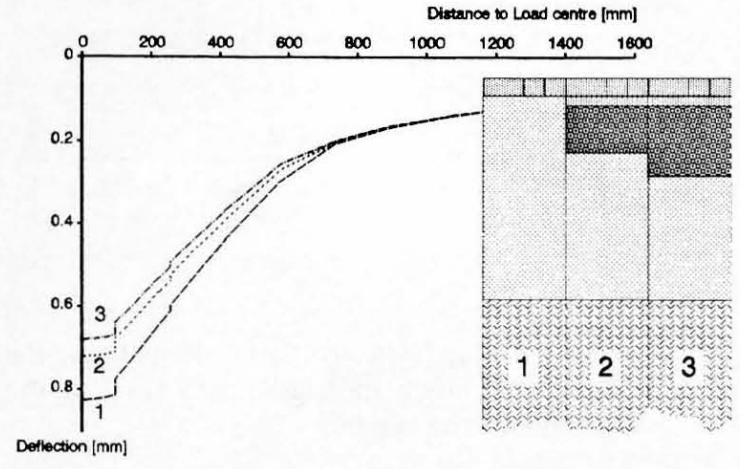


Fig. 10 Deflection bowls as calculated for the three concrete block pavements.

In all cases the pavement structure is placed over the subgrade (E= 60 MPa, μ= 0.45). The three pavement structures are all loaded by a 50 kN circular wheel load with a radius of 150 mm.
In figure 10 the calculated deflection bowls are presented. Again the calculated deflection bowls clearly show the discontinuous character of the concrete block layer.
As might be expected the deflections decrease as the thickness of the base layer increases. Major differences in the computed deflection bowls are however not found.
In the joints between the blocks, joint forces develop as a result of the pavement deflection, i.e. block rotation and translation.

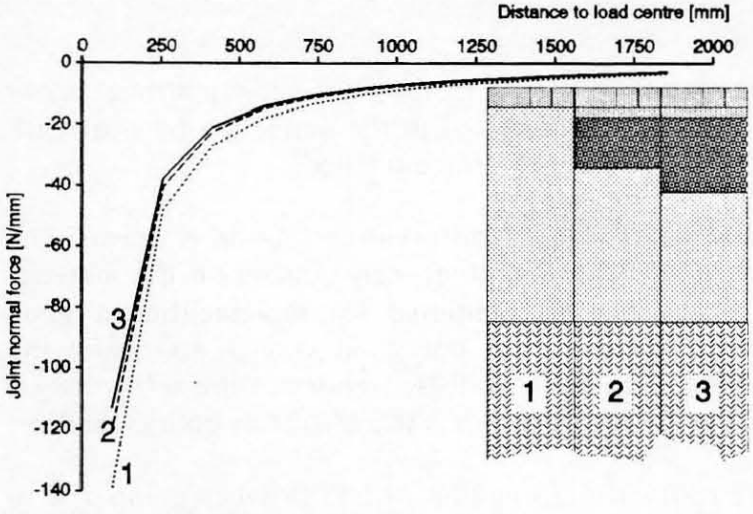


Fig. 11 Calculated joint normal forces.

In figure 11 the joint normal forces that develop in the three CBP discussed here are presented. Again the effects of the unbound base layer are limited. The joint normal forces in the CBP with a 300 mm base are about 15 % smaller than the joint normal forces in the CBP without a base.
The edge restraint of a CBP is forced outward by these joint normal forces. Although the finite element model is axial symmetric and thus does not have a representation of an edge restraint it is expected that the expanding forces on the edge restraint will also decrease with about 15% as a result of applying a 300 mm base course.

As discussed earlier the goal of this research is to explain and understand the development of permanent strain in a block pavement. For this purpose the $\sigma_1/\sigma_{1,f}$-ratio is most important [1]. In figure 12 this ratio as determined in the centre of the model is presented for the three CBP discussed here.

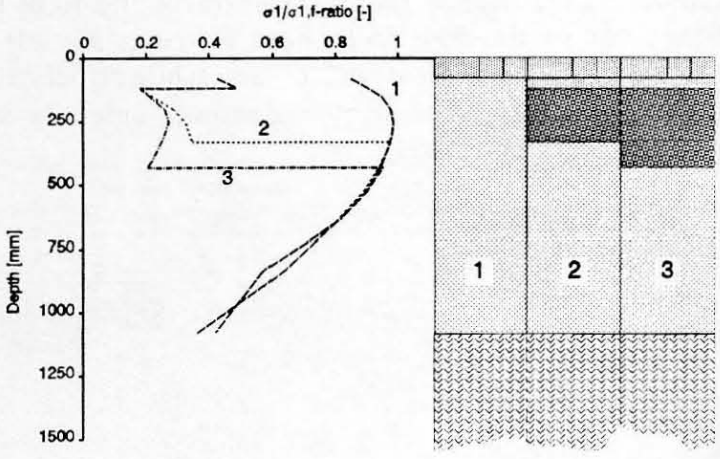


Fig. 12 The $\sigma_1/\sigma_{1,f}$-ratio over the height of the concrete block pavement structure in the centre of the model.

As shown by figure 12 the sub-base sand is subjected to large stress-ratios. In case of a pavement without a base layer the $\sigma_1/\sigma_{1,f}$-ratio is very close to 1 at a depth of about 250 mm. At this depth the base layers are situated. In these base layers much smaller $\sigma_1/\sigma_{1,f}$-ratios are computed (<0.4). As shown by figure 12 the bedding sand above the base layers also shows relatively small $\sigma_1/\sigma_{1,f}$-ratios when compared to the pavement without a base.
Of course the $\sigma_1/\sigma_{1,f}$-ratio is not only known in the centre of the model but throughout the substructure of the block pavement. In [1] it is discussed how this ratio makes it possible to determine the permanent strain development in the substructure of a CBP.

4. Conclusions

The granular materials used in the substructure of Dutch CBP show a strong stress-dependent resilient behaviour. If one is only interested in the deflection bowl of CBP it does not really matter how this material behaviour is modelled.

Considering the stresses in the substructure a total different conclusion is drawn. The calculated stresses in the substructure of a pavement strongly depend on the material model applied. Realistic stresses can only be computed by implementing a good material model. The widely used Mr-Θ model is not good enough to model the resilient behaviour of sands. In order to come to realistic stresses in the substructure, sands have to be modelled by a model that distinguishes the effects of both σ_3 and σ_1.

The effects of a base layer on the resilient deformation of a CBP when compared to a CBP without a base course are limited. A base course however strongly decreases the $\sigma_1/\sigma_{1,f}$-ratio at a depth of about 250 mm underneath the surface. At this depth the sand in the CBP without a base course shows the largest $\sigma_1/\sigma_{1,f}$-ratio. A base course thus strongly increases the quality of a CBP.

5. Acknowledgements

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Fabes	Dutch Association of Concrete Paving Block Manufacturers,
NVWB	Dutch Association of Road Contractors

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