THE SHEAR FAILURE MECHANISM IN CONCRETE BLOCK PAVEMENTS WITH A SAND SUB-BASE ONLY

M. Huurman, L. J. M. Houben, A. A. A. Molenaar and A. W. M. Kok
Delft University of Technology, Delft, The Netherlands

SUMMARY

The Delft University of Technology is performing a research program to gain more knowledge about the structural behaviour of concrete block pavement structures. The ultimate goal of this research program is to develop a method to predict the permanent deformations in concrete block pavement structures due to traffic loads, which means the development of rutting and the variation of rutting in the longitudinal direction.
In order to be able to predict the permanent deformation of a concrete block pavement structure a good understanding of the resilient behaviour of the structure is required. A straight forward application of the known methods for the calculation of stresses in concrete block pavement structures results in stress levels that are higher than the stresses at the point of failure. Therefore an in depth analysis of these available methods and especially the available models to characterise the material response is needed in order to arrive to acceptable predictions. In this paper a modified model for the resilient behaviour of sand just before failure is presented. Using this modified material model, a new insight in the resilient behaviour of a concrete block pavement is gained. Local failure does not occur so that the calculated stresses can be used to predict permanent deformation.

1. INTRODUCTION

Concrete block pavement structures are widely used in The Netherlands. These structures are often constructed without a base. The substructure is made of a 500 to 1000 mm sand sub-base only, placed on the subgrade which has a dynamic Young's modulus of elasticity between 25 and 150 MPa. The concrete block layer mostly comprises 70 to 90 mm thick rectangular concrete paving blocks with horizontal dimensions of 211 x 105 mm^2, placed in herringbone bond. It may be clear that such structures are very weak, nevertheless they are widely used.

Calculations performed using different methods of determining stresses, (Boussinesq/Odemark, Burmister, multi layer calculations, Finite Element Methods (FEM) linear or non-linear), all point in the same direction. The calculated stresses are much higher than the failure stresses (stresses at the point of failure) so that failure has to be expected. Since failure very often does not occur in real pavement structures, there is an obvious mismatch between the calculations and reality.
Because of the fact that the calculated stresses are higher then the failure stresses it is not possible to determine the permanent deformation on the basis of stresses. This explains why the calculated or measured resilient deformation is often used to determine permanent deformation. It has however been proven that there is no direct correlation between resilient and permanent deformation (1). The only correlation between resilient and permanent deformation behaviour is formed by stresses, which implies that the described problem keeps us from a universal solution of the prediction of permanent deformation of a concrete block pavement structure.

It is most likely that a concrete block pavement structure has a special way of bearing traffic loads. It prevents failure by re-distribution of stresses. This re-distribution of stresses is caused by the fact that when

failure is approached, granular materials become weaker in stead of stiffer and other parts of the structure take over the carrying of the load. For Eastern Scheldt sand this weakening could already be proved. It is strongly believed that base materials like crushed masonry, crushed concrete or lava will show similar behaviour so that it is most likely that it will become possible to determine permanent deformation of a concrete block pavement on the basis of stresses within the structure itself.

This paper explains the proposed material model, and shows that this model meets the results of triaxial tests. It explains that stresses determined on the basis of the proposed material model remain smaller than the failure stresses, as is the case in real pavement structures. Furthermore calculations of the rut depth on the basis of stresses show that the proposed material model leads to stresses which make it possible to make realistic rut depth predictions.

2. RESILIENT BEHAVIOUR

It is proved that elastic strains, which are easy to determine, do not correlate directly with permanent strains in granular materials (1). A good correlation between resilient behaviour and permanent strains is only obtained when stresses are considered, see chapter 4.
The above mentioned facts tell us that the only proper way to predict permanent deformation of a pavement structure is to calculate stresses caused by a traffic load. Using these stresses permanent strains can be determined. Integration of the permanent strains over the total height of the pavement structure (and the upper part of the subgrade if stresses within the subgrade are high enough to cause permanent strains) gives the rut depth. It is clear that variation of layer thicknesses, material parameters, traffic loads (dynamic effects), etc. will explain variation of the rut depth.
This method of predicting permanent deformation of the granular substructure of a pavement structure was earlier used for asphalt pavements by Barksdale and showed very promising results (2). Because of the limited load spreading effect of a concrete block layer, calculated stresses (using the traditionally methods) within concrete block pavement structures are much higher than the failure stresses of the used materials. This problem has made it impossible to predict rutting in concrete block pavements in the way it should be predicted. To solve this problem a new material model is proposed.

The resilient behaviour of granular materials is mostly determined using a triaxial apparatus, which is extensively discussed elsewhere (1). The resilient behaviour of granular materials is expressed as a function of the stresses within the material itself. When the stresses become higher, the material becomes stiffer. This is for instance explained by the widely used Mr-θ model introduced by Brown and Pell (3). This model, extended with a relationship expressing the stress-dependent Poisson's ratio, contains the following equations:

$$Mr = kl\,\theta^{k2} \qquad [1]$$

$$\mu = c + d\,(\sigma c/\sigma 3) \qquad [2]$$

where:
- Mr = resilient modulus
- θ = sum of the principal stresses
- kl,k2 = material parameters
- μ = Poisson's ratio
- $\sigma 3$ = confining stress applied on the sample
- σc = cyclic deviator stress applied on the sample
- c,d = material parameters

The mentioned Mr-θ model does not consider al,f which is the first principal stress at the point of failure. This al,f can be calculated from Mohr's failure curve:

$$\sigma 1,f = ((1 + \sin \phi)\, \sigma 3 + (2\, c \cos \phi)) / (1 - \sin \phi) \quad [3]$$

where: $\sigma 1,f$ = first principal stress at failure
$\sigma 3$ = third principal stress (confining stress)
ch = angle of internal friction
c = cohesion

Since failure is not considered the mentioned model (equation 1 and 2) makes granular materials stiffer as σ1 becomes larger. Given a certain σ3 materials keep getting stiffer as al becomes larger, so that a material far beyond the point of failure becomes far stiffer than the same material just before failure. This is obvious since θ is mainly determined by σ1.
It is clear that when failure is approached a sample must become weaker and weaker. At the point of failure the chord Mr-modulus becomes nil, so that the secant Mr-modulus, which is discussed here, becomes undefinable by stresses. To take this behaviour into account the σ1/σ1,f ratio must be considered. A new material model is proposed to do this. In this model Mr and μ are determined for a σ1/σ1,f ratio which is almost 0. These values are called Mr1 and μ1; μ1 is a constant while the function for Mrl is similar to equation 1, however dependent on σ3:

$$Mr1 = k3\, \sigma 3^{k4} \quad [4]$$

where: Mr1 = resilient modulus at very low σ1/σ1,f ratio, depending on σ3
k3,k4 = material parameters
σ3 = confining stress

To determine the effect of the σ1/σ1,f ratio "n", a function f(n) is introduced:

$$f(n) = \frac{(1/\sqrt{[1- n]}) - 1}{b} \quad [5]$$

with

$$b = \frac{(1/\sqrt{0.07}) - 1}{0.5\, (1 - \mu 1)} \quad [6]$$

where: σ1 = first principal stress
$\sigma 1,f$ = first principal stress at failure, see equation 3
μ1 = Poisson's ratio at very low $\sigma 1/\sigma 1,f$ ratio

Using this relation Mr and μ can be expressed as follows:

$$Mr = Mr1\, [1 - f(n)] \quad [7]$$

$$\mu = \mu l + f(n) \quad [8]$$

where: Mr = resilient modulus
μ = Poisson's ratio

Mr1 = resilient modulus at low σ1/σ1,f ratio determined by equation 4
μ1 = Poisson's ratio at very low σ1/σ1,f ratio
f(n) = relationship expressing the influence of the σ1/σ1,f ratio on the stiffness of a material, equations 5 and 6

This model determines Mr as a function of σ3, the degree of failure determines whether Mr should become smaller. When al/al,f is small Mr is almost equal to Mr1, but when σ1/σ1,f becomes larger than 0.8 the effect of f(σ1/σ1,g) becomes larger, as can be seen in figures 1,2 and 3.

Figures 1 and 2 show that the proposed model meets the results of triaxial tests performed on Eastern Scheldt sand far better than the Mr-θ model. When σ3 is set to be constant and al becomes larger the material becomes weaker instead of stiffer. At very high σ1/σ1,f ratio's Mr is represented by an almost vertical line in the Mr-θ plot (figure 2) so that Mr becomes almost undefinable by stresses. It will be clear that this behaviour has its influence on the way how a concrete block pavement structure with a sand sub-base only bears a load, as will be discussed in the next chapter.

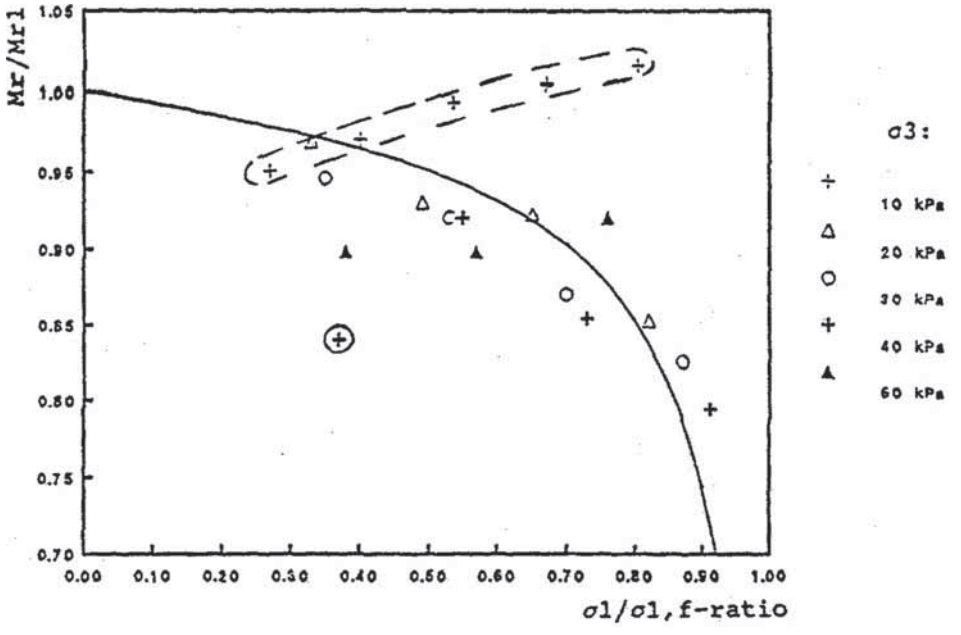


Figure 1: The results of triaxial tests performed on Eastern Scheldt sand in combination with the proposed relation f(n), equations 5 and 6. The measurements are performed at different confining stresses as shown at the right side of the plot. Some strange data points are marked so that they can easily be found in figure 2.

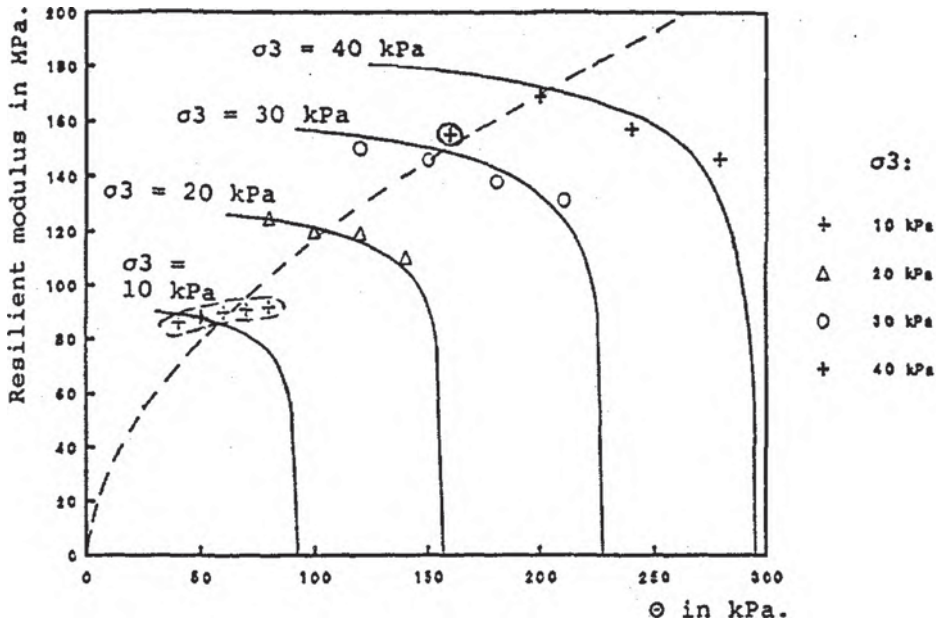


Figure 2: Mr-moduli as determined with the Mr-θ model (scattered line) and Mr-moduli determined with the proposed model (continuous lines) in combination with measurements performed at different confining stresses.

As figures 1 and 2 show, Eastern Scheldt sand shows an increasing stiffness as σ1/σ1,f becomes higher when the confining stress is 10 kPa. It is assumed that this is caused by the fact that the confining stress in a dynamic triaxial test is hard to keep constant when it is small.

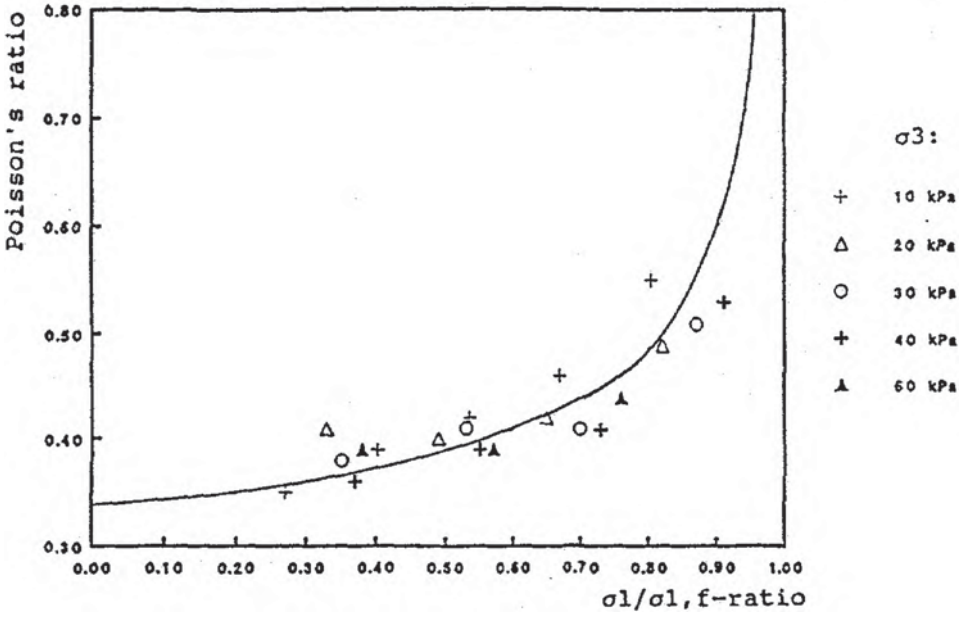


Figure 3: Measured Poisson's ratios as a function of σ1/σ1,f compared with the proposed material model.

3. STRESSES CAUSED BY TRAFFIC LOADS

Since the proposed material model is not yet linked with a Finite Element Model, the results of an earlier non-linear Finite Element Model of a concrete block pavement are considered. This model was based on the earlier mentioned Mr-0 model and represented a pavement structure consisting of 80 mm thick

rectangular concrete paving blocks with horizontal dimensions of 211 x 105 mm². This concrete block layer was placed on a 1550 mm thick sand sub-base. The subgrade had a dynamic Young's modulus of elasticity of 40 MPa. The model was loaded by a 50 kN wheel load (6).
When the model was calculated using a constant Poisson's ratio of 0.35, the maximal ul/ul.f ratio was calculated to be 2.9 at about 270 mm beneath the concrete block layer. This means that the calculated stresses where about 3 times higher than the failure stresses. To reduce the $\sigma 1/\sigma 1,f$ ratio the elements where $\sigma 1/\sigma 1,f$ was higher than 1 were given a Poisson's ratio of 0.49. This did indeed reduce $\sigma 1/\sigma 1,f$ since $\sigma 3$ became larger. At 270 mm beneath the concrete block layer $\sigma 1/\sigma 1,f$ now was calculated to be 1.4, which is however still too high.

To get an impression of the results which would be obtained when the proposed material model for sands would have been used, the stresses in different elements were recalculated. This was done by assuming that resilient strains will not differ when the new material model is linked with the Finite Element Model. This assumption may certainly not be called strange since the deflections to be calculated remain the same.
These recalculations shown that a concrete block pavement with a sand sub-base bears a load in a completely different way than calculations, using material models which don't consider failure, tend to show. Furthermore, the $\sigma 1/\sigma 1,f$ ratio at 270 mm beneath the concrete block layer, which remains the maximal $\sigma 1/\sigma 1,f$ ratio, becomes smaller than 0.95.
Figure 4 shows that stresses are re-distributed to prevent a structure from failure. A wheel load is not only supported by the material underneath the load itself but also by material at a certain horizontal distance from the load centre. Failure in the material underneath the load is approached very close, so that this material becomes weak. The surrounding material is further from failure so that this material becomes relatively stiffer, as shown in figure 5. The relative stiffer parts in a structure tend to attract stresses, so that the weaker parts are partly unloaded.
In order to determine if the calculated stresses will lead to real rut depth, calculations of the rut depth are made and discussed in the next paragraph.

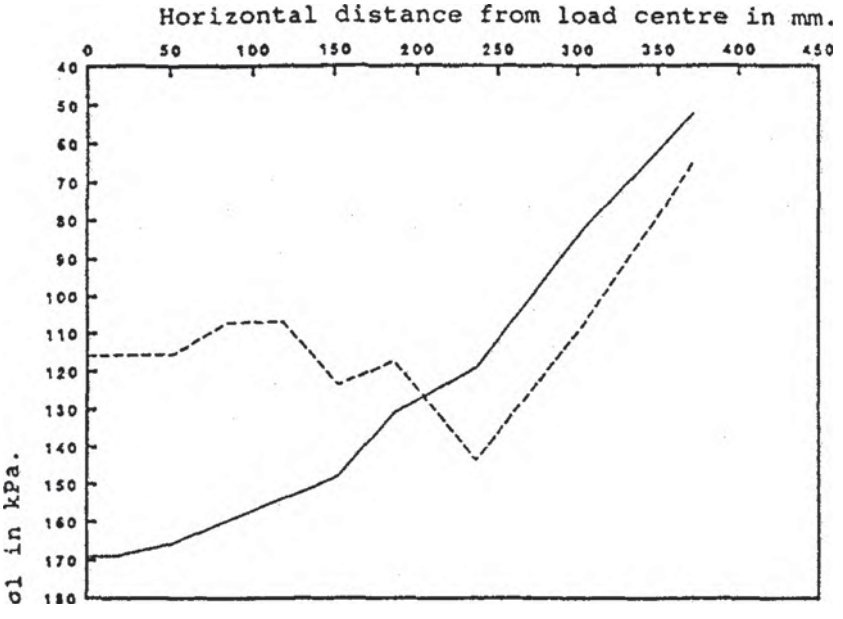


Figure 4: Stresses as calculated by a non-linear model at a depth of 268 mm underneath the concrete block layer (continuous line) compared with recalculated stresses determined on the basis of the new material model (discontinuous line).

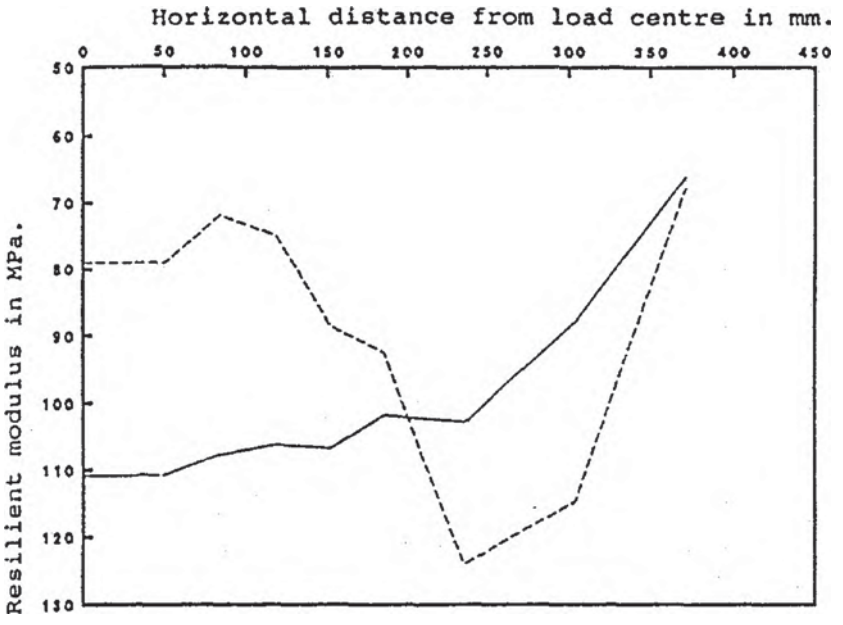


Figure 5: The influence of the proposed model on stiffnesses. The continuous line represents the stiffnesses as calculated by the Mr-θ model, while the discontinuous line represents the stiffnesses as determined with the proposed material model.

The effects of the proposed material model on the stress distribution, as shown in figure 4 is supported by soil pressure measurements performed in Amsterdam (7,8). These measurements where performed on a structure with a 190 mm thick crushed concrete base. Within the 60 mm thick bedding layer the measured pressures caused by a 50 kN load were distributed the same way the presented calculations show, see figure 6. At some points underneath the base, the distribution of the measured pressures had a shape which equals the calculated shape too. This implies that even underneath the weaker spots of a 190 mm thick base, local failure is prevented the way the presented material model shows.

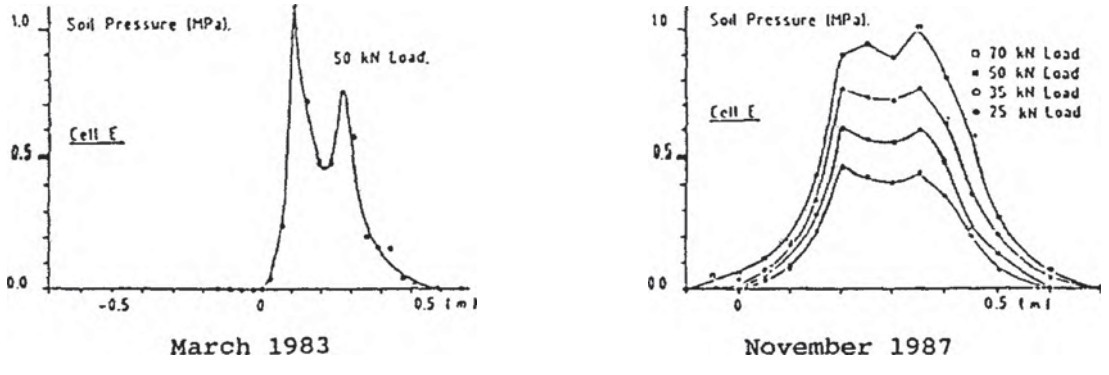


Figure 6: Measured vertical pressure within the bedding layer of a based concrete block pavement structure due to falling-weight loads (7,8).

4. CALCULATION OF PERMANENT DEFORMATION ON THE BASIS OF STRESSES

To determine whether the proposed method can work a simplified analytical model is created. This model is based on Boussinesq/Odemark.
A concrete block pavement structure was divided into 50 mm thick layers. The concrete block layer is supposed to be a layer with properties determined by Fuchs (7):

$$E = 918 \log N - 1719 \text{ [Mpa]} \qquad [9]$$

with

$$\mu = 0.25$$

where: E = Young's modulus
N = number of equivalent 80 kN standard axle load repetitions
μ = Poisson's ratio

The material underneath the concrete block layer was modeled in such a way that Mr could not be greater then Mr determined for an assumed $\sigma 1/\sigma 1,f$-maximal stress state. The input parameter $\sigma 1/\sigma 1,f$-maximal hereto determines in what proportion failure is prevented by a certain concrete block pavement structure. If stresses at a certain depth were higher than $\sigma 1/\sigma 1,f$-maximal, stresses were supposed to be equal to the stresses in the $\sigma 1/\sigma 1,f$-maximal state. In other words the degree of stress re distribution was set to be an input parameter so that the permanent deformations could be calculated for a certain supposed stress distribution.
Using this model it becomes possible to determine the stress distributions which will lead to real rut depth.
To determine the rut depth the stresses caused by a 40 kN wheel load are calculated for each log N = 0.5.
Given the calculated stresses, permanent strain in the material can be determined with the following relation (1):

$$\text{Log } \varepsilon a,p = a + b \text{ Log } N \qquad [10]$$

where: $\varepsilon a,p$ = permanent axial strain determined with a triaxial apparatus
a,b = material parameters depending on $\sigma 1/\sigma 1,f$
N = number of load repetitions

When equation 10 is used, the effects of compacting the structure during construction are not considered. These effects of the compaction efforts are however expressed in a certain number of load repetitions "N0". Equation 10 now becomes:

$$\varepsilon a,p = 10^{(a + b \log [N + N0])} - 10^{(a + b \log N0)} \qquad [11]$$

where: ea,p = permanent axial strain
a,b = material parameters depending on $\sigma 1/\sigma 1,f$
NO = number of load repetitions expressing the compaction efforts
N = number of load repetitions

Using the explained simplified model in which the prevention of failure is taken into account by a supposed stress distribution, several constructions have been calculated. The values of N0 and $\sigma 1/\sigma 1,f$-maximal are

chosen in such a way that the calculated progress of rutting equals the development of rutting as determined in order to develop the Dutch design method for concrete block road pavements (8).
The results of these calculations point in the same direction as the earlier discussed re-calculated stresses. Depending on the concrete block pavement structure al/al,f-maximal was determined to be 0.83 to 0.924. N0 was determined to be 200 to 11,000, also depending on the concrete block pavement structure. The determined σ1/σ1,f-maximal values show that constructions on a weak subgrade are not very capable in preventing failure (high σ1/σ1,f-maximal), which is exactly what must be expected. The N0-values showed that concrete block pavement structures on a weak subgrade can not be compacted well (low N0) which again meets the expectations, see table 1.

Thickness of sand sub-base	Dynamic Young's modulus of the subgrade	σl/σl,f-maximal	N0
700 mm	40 MPa	0.924	200
700 mm	60 MPa	0.893	3,200
700 mm	100 MPa	0.87	8,000
700 mm	140 MPa	0.832	11,000
1500 nun	40 MPa	0.895	900

Table 1: The values of N0 and σ1/σ1,f-maximal determined for different concrete block pavement structures.

5. CONCLUSIONS

The only proper way to determine permanent strains in concrete block pavement structures is to take into account the effect of re-distribution of stresses in a concrete block pavement structure. Up till now this approach has not been used. Dynamic triaxial tests however show that Eastern Scheldt sand becomes weaker when failure is approached. Since the stiffness of a granular material is normally expressed as a function of G, it becomes clear that the existing material models (Mr-θ type) do not consider failure. This implies that these models are not accurate enough to model the resilient behaviour of granular materials, especially when failure is approached very close, which is the case in concrete block pavement structures. When Mr for sands is expressed as a function of σ3 and σ1/σ1,f the results of dynamic triaxial tests are far better approached. Using a new material model which considers failure it could be proved that re-distribution of stresses occurs so that failure is prevented. Calculated σ1/σ1,f ratios caused by a 50 kN wheel load within a structure with only a 1550 mm thick sand sub-base on a subgrade with a Young's modulus of 40 MPa became 0.88 to 0.94. These ratios were larger than 1 when the Mr-θ model was used.

The conclusion of the presented calculations must therefore be that there is a theoretical correct way to predict rutting and variation of rutting in concrete block pavement structures. By calculating stresses in the structure at several points in N (progressive stiffening) and integrating the permanent strains caused by these stresses over the height of a structure, real development of rut depth can be obtained.

Facts:

1 Permanent strains in granular material as determined in laboratory triaxial tests make it possible to describe the development of rutting in a concrete block pavement structure if failure is approached very close and the effect of compaction is considered.

2 When the weakening of a granular material approaching the point of failure, as for Easter Scheldt sand determined with laboratory tests, is considered, a new material model is obtained. When this new model is used to calculate stresses within a concrete block pavement structure $\sigma 1/\sigma 1,f$ ratios lower than 0.95 were obtained.

3 There is no direct correlation between resilient strains and permanent strains of granular materials. A correlation between stresses and permanent strains is however found to exist (1). Furthermore the resilient vertical compression of a wheel loaded layer on a poor subgrade is less then the vertical compression of the same layer loaded by the same load on a stiff subgrade. This implies the total construction as well as the individual granular materials do not show a correlation between resilient deformation and permanent deformation.

It is shown that accurate calculations of the stress distribution are needed in order to determine the permanent deformation of a concrete block pavement A research program which has been started at the Delft University of Technology will therefore give a lot of attention to the further development of a model of the resilient behaviour of a concrete block pavement structure. The ultimate goal of this modelling is to obtain stresses which are precise enough to determine the rut depth and the variation of rut depth.

6. ACKNOWLEDGEMENTS

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