RUT DEVELOPMENT IN CONCRETE BLOCK PAVEMENTS DUE TO PERMANENT STRAIN IN THE SUBSTRUCTURE

M. Huurman
Delft University of Technology
Delft, The Netherlands

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, as discussed elsewhere [1].
In this paper the properties of eight Dutch sands and four Dutch unbound base course materials considering the development of permanent strain are discussed. It is shown that these properties can be used to determine the development of permanent strains throughout the CBP substructure. Insight into the development of rutting is now obtained. It is shown that CBP without a base develop large substructure permanent vertical strains at a depth of about 250 mm underneath the pavement surface. By applying a base course these large permanent vertical strains are reduced substantially, resulting in a far greater design life.

1. Introduction

The goal of the research project 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 wheel loadings.
This paper is strongly related to the paper "Resilient behaviour of Concrete Block Pavements and the Granular Materials used in the Substructure" [1]. In that paper the development of wheel load introduced substructure stresses is discussed. These stresses, calculated taking into account the resilient behaviour of the applied granular materials, are used in this paper to determine the permanent strain development throughout the substructure. On the basis of this permanent strain development rutting is explained.

2. Material Behaviour

2.1 *Introduction*

In [1] the resilient behaviour of eight Dutch sands and four Dutch base course materials is discussed. Also the cohesion C and the angle of internal friction ϕ of these materials are discussed in that paper.
In this paper the development of permanent strain in the same materials is discussed. Again the materials are named after the in-service CBP from which the materials are taken.

2.2 *Permanent strain development*

By means of a cyclic loading triaxial test, discussed elsewhere [1], insight into the development of permanent strains within a granular material can be obtained. For this purpose a sample of a granular material is first confined by the confining pressure, σ_{conf}. Hereafter a small static vertical stress is applied, σ_{stat}. By means of an actuator a halfsine-shaped cyclic stress, σ_{cycl}, can be applied to the sample. For this research all permanent strain triaxial tests are performed at a σ_{conf}-level of 12 kPa. As a result of σ_{cycl} the sample will show some deformation, i.e. strain. The major part of this strain is resilient, a minor portion of the strain however shows a permanent character, see figure 1.

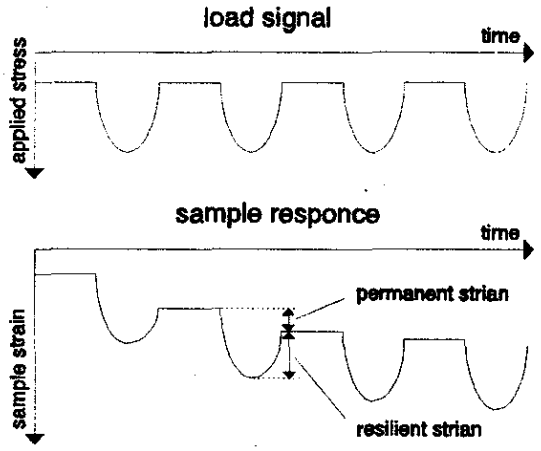


Fig. 1 Strain development in a granular material during repeated cyclic loading.

During the repeated cyclic load triaxial test the sample is subjected to up to 1,000,000 load repetitions. During these load repetitions the sample keeps accumulating permanent strain. It might be clear that the rate of permanent strain accumulation

strongly depends on the cyclic stress applied to the sample. The larger this stress the stronger the permanent strain accumulation.
To describe the stress dependent permanent strain development, the stresses applied to a sample are translated to the $\sigma_1/\sigma_{1,f}$-ratio. In this ratio $\sigma_{1,f}$ is the first principal stress at failure based on the C and ϕ of the material (equation 7 in [1]). σ_1 equals the actual applied first principal stress (equation 3 in [1]).
For this research the development of permanent strain is described by means of equation 1. This equation is frequently used to describe the creep curves measured on asphalt mixes [2].
As mentioned earlier the permanent strain development strongly depends on the applied stresses. The parameters A, B, C and D in equation 1 thus do not only depend on the material that is tested, but also on the stresses at which the triaxial test is performed. As a result the parameters A, B, C and D are stress dependent. On the basis of the $\sigma_1/\sigma_{1,f}$-ratio it is found that the stress dependency in the parameters A, B, C and D can be described by simple equations, equation 2, 3, 4 and 5.

$$\epsilon_{perm} = A\left(\frac{N}{1000}\right)^{B} + C\left(\exp\left(D\frac{N}{1000}\right) - 1\right) \qquad (1)$$

$$A = a1\left(\frac{\sigma_1}{\sigma_{1f}}\right)^{a2}, \qquad B = b1\left(\frac{\sigma_1}{\sigma_{1f}}\right)^{b2}, \qquad (2),(3)$$

$$C = c1\left(\frac{\sigma_1}{\sigma_{1f}}\right)^{c2}, \qquad D = d1\left(\frac{\sigma_1}{\sigma_{1f}}\right)^{d2} \qquad (4),(5)$$

Where:

ϵ_{perm}:	permanent strain [-]
N:	number of applied load cycles [-]
a1,a2 b1,b2, c1,c2, d1,d2:	model parameters [-]

The equations given above apply for both the radial and axial permanent strain.

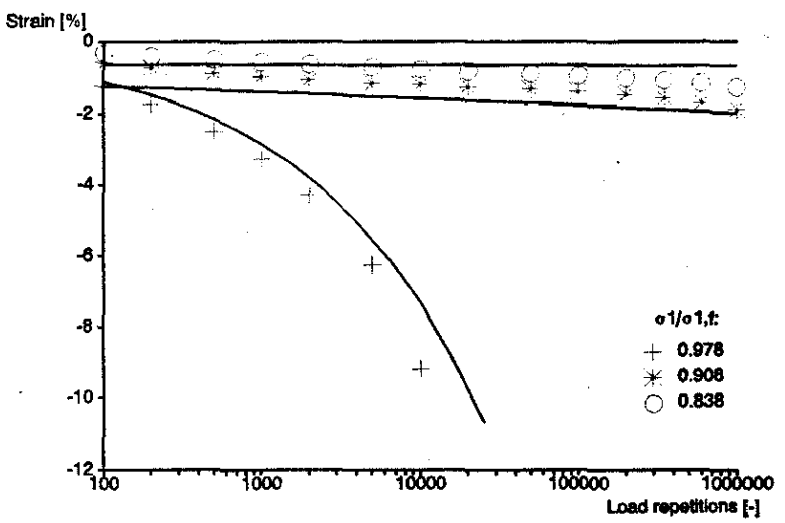


Fig. 2 Development of permanent axial strain in the Zaanweg sub-base sand.

As an example, in the figures 2 and 3 the permanent strain development in the Zaanweg sub-base sand is presented. In these figures not only the measured data but also the discussed model (equations 1 to 5) is plotted. As shown by the figures 2 and

3, the Zaanweg sand can be subjected to stresses that result in a very high $\sigma_1/\sigma_{1,f}$-ratio without showing rapid failure. Even if subjected to a cyclic stress that results in $\sigma_1/\sigma_{1,f}$-ratio of about 0.98 the sample needs 10,000 load repetitions in order to show a permanent axial and radial strain of about 10%.
If the sand is subjected to a cyclic stress introducing a $\sigma_1/\sigma_{1,f}$-ratio of about 0.91, only minor permanent strains of about 2% are measured after 1,000,000 load repetitions.

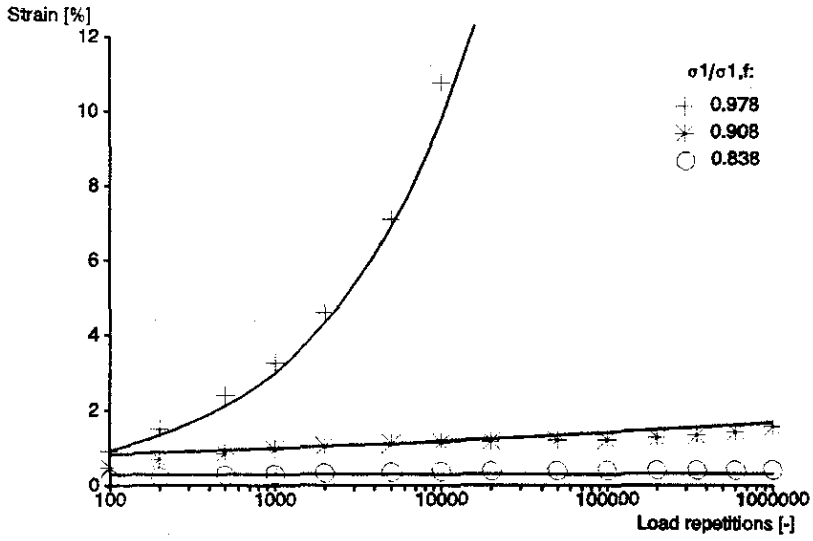


Fig. 3 Development of permanent radial strain in the Zaanweg sub-base sand.

Similar results are obtained for the other sands, see tables 1 and 2. In these tables the values of the model parameters c1, c2, d1 and d2 are not given. For the sands these parameters equal zero. The model thus describes the permanent strain development in a sand by a straight line on a log-log scale.

	a1 % [-]	a2 [-]	b1 [-]	b2 [-]	r^2
Allanstraat sand	-5.01	16.40	0.30	20.07	0.85
Weiver sand	-2.72	6.64	0.27	6.05	0.99
Baarsweg sand	-2.54	5.26	0.07	0.00	0.86
Max Havelaarweg sand	-4.97	4.96	0.31	13.27	0.82
Zaanweg sand	-3.53	9.59	0.76	27.80	0.80
Cor Bruinweg sand	-5.19	5.93	0.14	1.13	0.88
Pascalweg sand	-5.46	6.61	0.29	6.06	0.95
Crusher sand	-2.76	2.69	0.18	0.50	0.70

Table 1: Model parameters found for the development of permanent axial strain in various Dutch sands.

	a1 % [-]	a2 [-]	b1 [-]	b2 [-]	r^2
Allanstraat sand	5.98	22.63	0.34	20.93	0.70
Weiver sand	3.13	9.16	0.32	8.67	0.98
Baarsweg sand	2.03	6.09	0.09	2.49	0.83
Max Havelaarweg sand	5.42	7.75	0.40	29.47	0.91
Zaanweg sand	4.20	14.89	0.90	25.44	0.95
Cor Bruinweg sand	6.82	8.29	0.16	2.38	0.91
Pascalweg sand	6.50	9.12	0.51	10.04	0.95
Crusher sand	3.23	4.88	0.18	0.34	0.705

Table 2: Model parameters found for the development of permanent radial strain in various Dutch sands.

On the four base course materials also permanent strain triaxial tests were performed. The results of these tests gave a completely different picture, see figures 4 and 5. As shown by these figures the Pascalweg base course material can not be subjected to cyclic stresses that result in a $\sigma_1/\sigma_{1,f}$-ratio which approaches 1. A firm permanent strain (permanent axial strain about 8%) is measured when the material is subjected to 1,000,000 load repetitions introducing a $\sigma_1/\sigma_{1,f}$-ratio of only 0.3. Similar results are obtained for the other three base course materials.

As discussed earlier a sand subjected to a $\sigma_1/\sigma_{1,f}$-ratio of about 0.3 would hardly develop any permanent strain. Compared to the sands a base course material thus develops firm permanent strains at much lower $\sigma_1/\sigma_{1,f}$-ratios.

The fact that base course materials develop permanent strains at much lower $\sigma_1/\sigma_{1,f}$-ratios than sands do, is easily misinterpreted. This certainly does not mean that base course materials accumulate permanent strain easier than sands do.

Base course materials show a larger C than the sands, [1]. As a result $\sigma_{1,f}$ of a base material is much higher than the $\sigma_{1,f}$ of a sand when both are subjected to the same σ_{conf}.

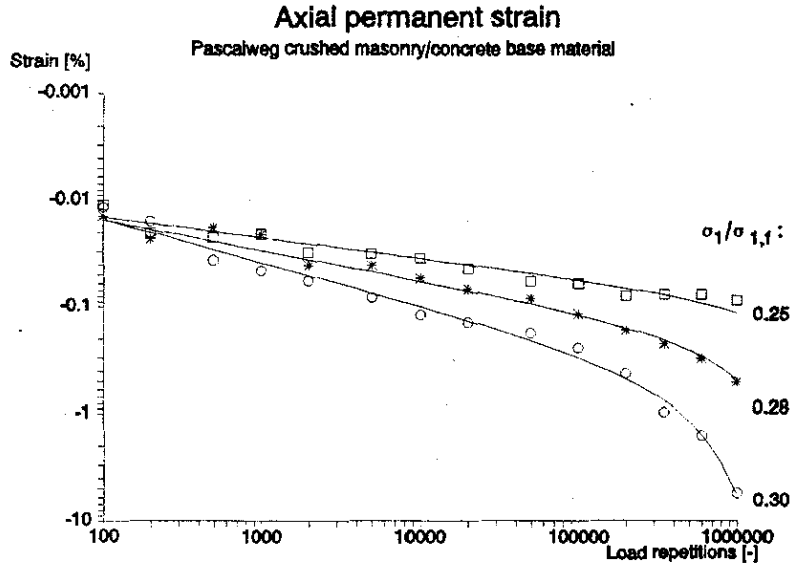


Fig. 4 Development of axial permanent strain in the Pascalweg base course material.

When compared to the sands, base course materials show another difference. Two of these materials showed a permanent strain development that can no longer be described by straight lines on a log-log scale. These materials show an increase in the rate of permanent strain accumulation after a certain number of load repetitions. This behaviour is described by the second term in equation 1, resulting in c1, c2, d1 and d2 values which are no longer equal to zero, see tables 3 and 4.

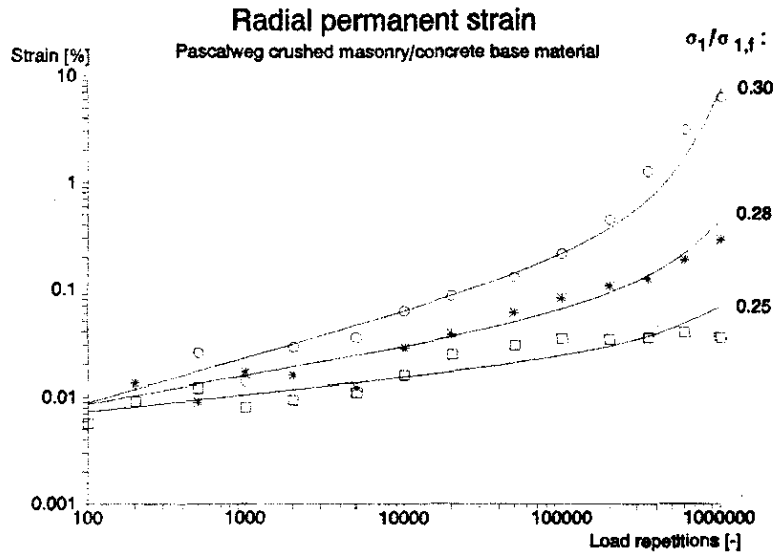


Fig. 5 Development of radial permanent strain in the Pascalweg base course material.

Base course material	A_1 % [-]	A_2 [-]	B_1 [-]	B_2 [-]	r^2 [-]
	C_1 % [-]	C_2 [-]	D_1 [-]	D_2 [-]	
Allanstraat	-492.95	10.51	14.62	4.95	0.864
	-	-	-	-	
M. Havelaarweg	-74.84	7.60	0.57	0.73	0.981
	-2.66	7.50	119.00	10.00	
C. Bruynweg	-2.58	3.58	2.77	2.99	0.972
	-	-	-	-	
Pascalweg	-1.24	2.90	52.09	4.04	0.985
	-1183.96	7.50	609.96	10.0	

Table 3: Model parameters found for the development of permanent axial strain in the four Dutch base course materials.

Base course material	A_1 % [-]	A_2 [-]	B_1 [-]	B_2 [-]	r^2 [-]
	C_1 % [-]	C_2 [-]	D_1 [-]	D_2 [-]	
Allanstraat	270.20	9.83	21.29	5.54	0.790
	-	-	-	-	
M. Mavelaarweg	403.82	9.93	0.46	0.12	0.977
	4.10	7.50	111.03	10.00	
C. Bruynweg	11.15	5.41	1.25	2.15	0.982
	-	-	-	-	
Pascalweg	3.48	4.19	227.36	5.27	0.964
	1419.36	7.50	640.15	10.0	

Table 4: Model parameters found for the development of permanent radial strain in the four Dutch base course materials.

3 Permanent strain development in the substructure

3.1 *Calculation procedure*

In [1] the stress dependent resilient properties of the eight Dutch sands and four Dutch base materials are discussed. In that paper also the cohesion and the angle of internal friction of these materials were given. On the basis of these properties the resilient behaviour of three CBP was computed. In all three cases the total pavement structure had a height of 1080 mm. The first layer had a thickness of 80 mm and represented the concrete block layer (E= 40,000 MPa, μ= 0.25). Pavement structure 1 only had a 1000 mm thick sand sub-base (stress dependent resilient behaviour of the Zaanweg sub-base sand).

The other two pavements had an unbound granular base layer. These pavements both had a 50 mm thick bedding sand layer (stress dependent resilient behaviour of crusher sand). Below this bedding layer the base course layer was situated (stress dependent behaviour of the Pascalweg base course material). In case of pavement structure 2 this base had a thickness of 200 mm, while structure 3 had a 300 mm thick base. These base layers were placed over a Zaanweg sand sub-base with a thickness of 750 and 650 mm respectively.

In all cases the pavement structure was placed over the subgrade (E= 60 MPa, μ= 0.45). The three pavement structures were all loaded by a 50 kN circular wheel load with a radius of 150 mm.

The calculations amongst others gave insight into the $\sigma_1/\sigma_{1,f}$-distribution throughout the substructure of the CBP. Also the direction of the principal stresses was known throughout the substructure. The combination of this information with the stress dependent permanent deformation behaviour of the relevant materials, figures 2, 3, 4 and 5 and tables 1, 2, 3 and 4 of this paper, makes it possible to compute the

permanent horizontal and vertical strain development throughout the substructure of the three pavement structures described above.

The material properties of the subgrade concerning permanent strain development are not known from laboratory research. However minor permanent strains are expected to develop in the subgrade. In order to also take into account these permanent subgrade strains use is made of a linear relation between resilient strain and permanent strain found for soils by Veverka [3]:

$$\epsilon_{perm} = \epsilon_{res} \times (a + b\ logN) \qquad (6)$$

where:

ϵ_{perm}: permanent strain [-]
ϵ_{res}: resilient strain [-]
a, b: model parameters [-]
N: number of load repetitions [-]

For a and b the average values found for soils by Veverka are implemented (a=0, b=0.7).
For the subgrade the vertical resilient strains are translated to the vertical permanent strain development using equation 6. In the substructure of the pavement the $\sigma_1/\sigma_{1,f}$-ratios are translated to permanent strains on the basis of the discussed material behaviour, equations 1 to 5. Given the directions of the principal stresses the vertical components of these permanent strains are determined.
The calculated vertical permanent strains are summed-up over the height of the substructure and the subgrade (finite element model, see [1]) and so give the permanent vertical displacement of the surface of the pavement. This displacement of-course depends on the number of load repetitions, N. In this way the development of permanent surface displacement as a function of N can be computed for any distance from the load centre.

These surface permanent deformation as well as the permanent strain on which they are based refer to the unrealistic situation in which absolutely no lateral wander occurs.
To explain how the effects of lateral wander are taken into account it is assumed that wheel loads only travel the pavement in two wheel tracks. These parallel wheel tracks are separated by a transversal distance of 300 mm, while the distribution of wheel loads over these wheel tracks is 70% against 30%, see figure 6.

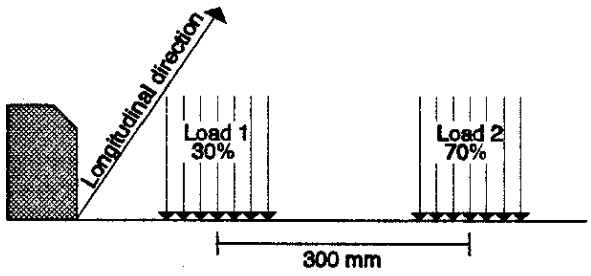


Fig. 6 Transversal load distribution.

The determination of the rut development under wheel load 1 as a result of both wheel load 1 and wheel load 2 is discussed hereafter. Hereto the rut development is computed at 0 mm and 300 mm from the centre of the load, see figure 7 sub-plots A and C. For computing the rut depth under wheel load 1, the development of the rut depth at a distance of 0 and 300 mm from the load centre are both simplified to straight lines over a distance of

for instance 10 load cycles, see sub-plot A.
The line found for the rut development at a distance of 300 mm is applied for 7 out of the 10 load repetitions (70%), while the line found for a distance of 0 mm is used for 3 out of 10 repetitions (30%). The rut depth after 10 load repetitions in the desired point (wheel track of load 1) is now known, see sub-plot B.

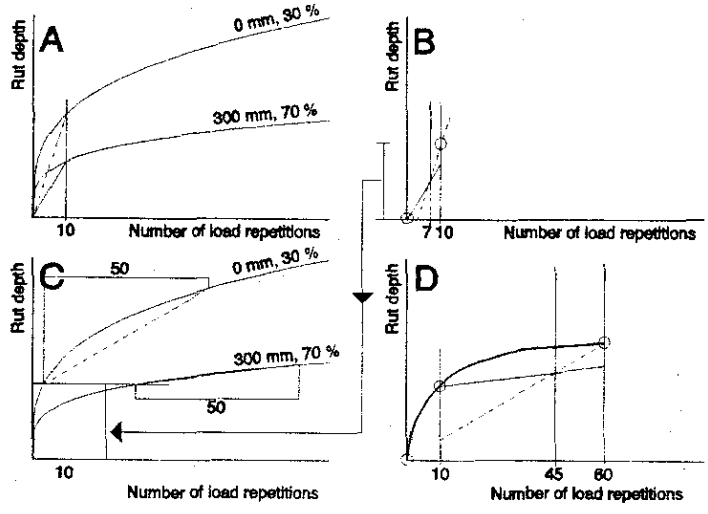


Fig. 7 Principal of taking into account the effects of lateral wander.

This rut depth is transferred to the original computed rut development, see sub-plot C. The effects of an additional 50 load repetitions are determined by again simplifying the curves by straight lines. This time the starting point of this simplification is no longer the origin, N=0, but a point at which the rut depth in the original curves equals the rut depth as computed for N=10, see sub-plot C. Again these lines are applied for 70% and 30% of the additional loads, i.e. 35 and 15 load repetitions respectively. Now the rut depth after 60 load repetitions is known, see sub-plot D.
Of course this algorithm can be repeated in N. Using the same algorithm the rut development in any point in the cross section can be determined as a result of wheel loads showing any transversal distribution (lateral wander).

In the previous example only the effects of lateral wander on permanent surface deformation are considered. On the basis of the same principal the effects of lateral wander on vertical permanent substructure strains can also be determined. As a result one gets insight into both the surface rut development and the permanent strains that cause these ruts.

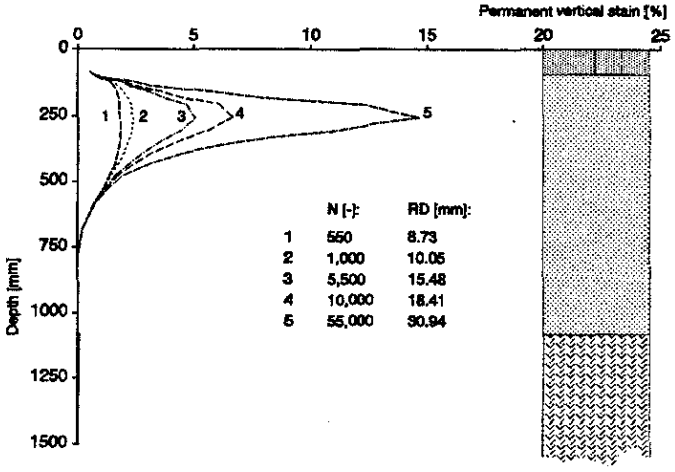


Fig. 8 Development of permanent vertical strain in the rut centre for the pavement structure with only a sand sub-base, σlw = 100 mm.

3.2 *Calculation results*

In figure 8 the development of permanent vertical strain in the rut centre is given for the concrete block pavement without a base. The standard deviation of lateral wander, σlw, in this computation is 100 mm, which is extremely small for normal road traffic

[4]. The load is a 50 kN wheel load with a radius of 150 mm.
Figure 8 shows that the largest vertical permanent strains develop at a depth of somewhat more than 250 mm. The rut depth reaches 15 mm, which is mostly considered as failure, after about 5,500 50 kN wheel load repetitions. The maximum permanent vertical strain at that moment is almost 5 %.
When σlw is doubled to 200 mm, the permanent strain development is reduced, and as a result the pavement will last longer. A 15 mm rut depth is now reached after about 10,000 wheel load repetitions, see figure 9. The maximum permanent vertical strain, at this moment is somewhat larger than 5 %.

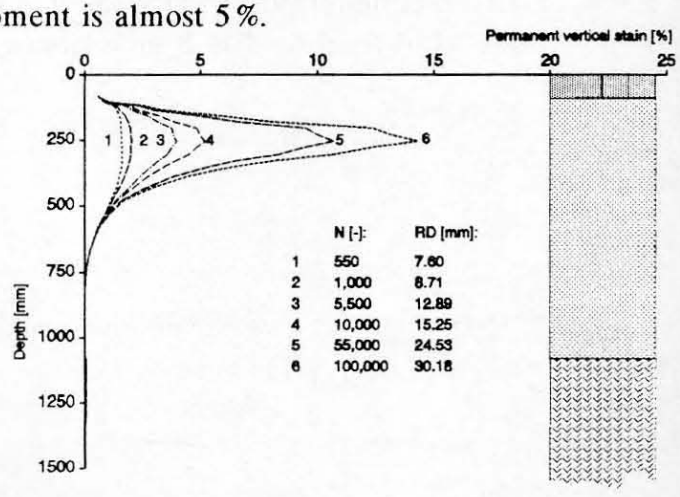


Fig. 9 Development of permanent vertical strain in the rut centre for the pavement with only a sand sub-base, σlw = 200 mm.

By applying a base layer the resistance of the pavement against permanent deformation increases. In case of the thin 200 mm base, larger permanent vertical strains develop at the bottom of the base. This process starts after about 100,000 load cycles and will eventually lead to failure of the pavement, see figure 10. A rut depth of 15 mm is now reached after about 600,000 load repetitions.
By increasing the thickness of the base layer to 300 mm, premature failure in this layer is prevented. In this case the largest permanent strains will develop at the top of the sand sub-base layer, see figure 11 (in figure 11 the scale of strain is much smaller than in the previous figures).
The block pavement with a 300 mm thick base layer shows a very strong resistance against permanent strain. After 1,000,000 load repetitions it only shows a rut depth of about 5.5 mm. More than half of this rut depth occurs during the first 1,000 load repetitions which implies that the pavement structure is very stable.

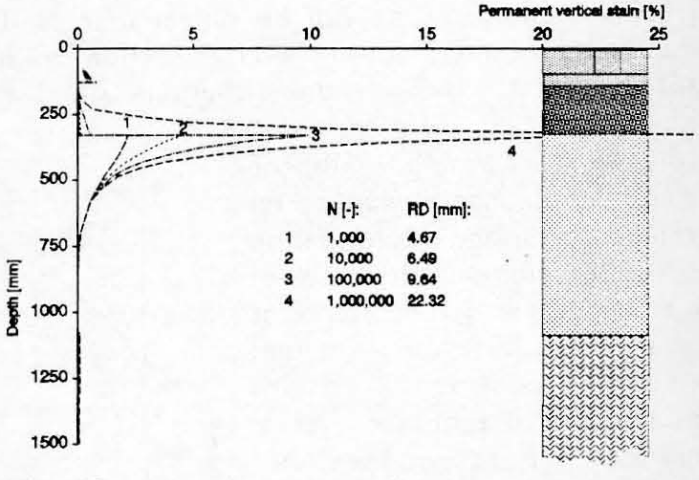


Fig. 10 Development of permanent vertical strain in the rut centre for the pavement structure with a 200 mm base and a sand sub-base, σlw = 200 mm.

In figure 12 the rut development for the three pavement structures is presented. In all cases σlw equals 200 mm. The figure shows that when no base layer is applied the

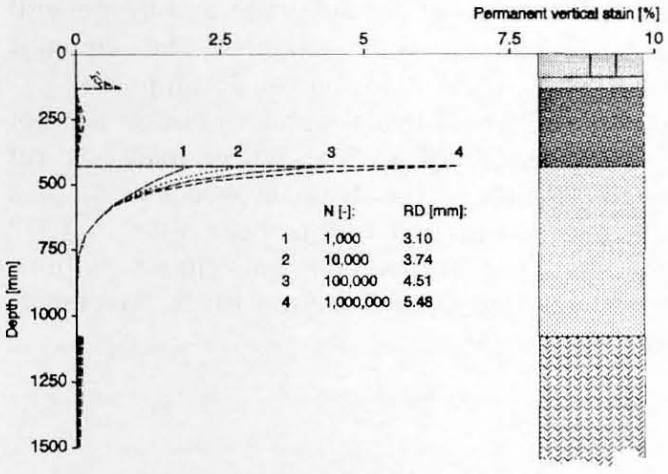


Fig. 11 Development of permanent vertical strain in the rut centre for the pavement structure with a 300 mm base and a sand sub-base, σlw = 200 mm.

pavement keeps accumulating damage rapidly until it reaches failure.

When a 200 mm base layer is applied the pavement will rapidly accumulate damage until it shows a rut depth of about 10 mm. After reaching this initial rut depth further accumulation of damage takes place at a much slower rate. As a result of the large initial rut depth drivers will experience a low quality concrete block pavement during the main part of the pavements life.

Figure 12 shows that applying a thick base does not only extent the design life of a pavement, but also the quality of this pavement during its life. In case of the pavement structure with a 300 mm thick base the initial rut depth is about 4 mm. During the largest part of the life of this particular pavement the accumulated damage only shows minor proportions, drivers thus experience a high quality concrete block pavement.

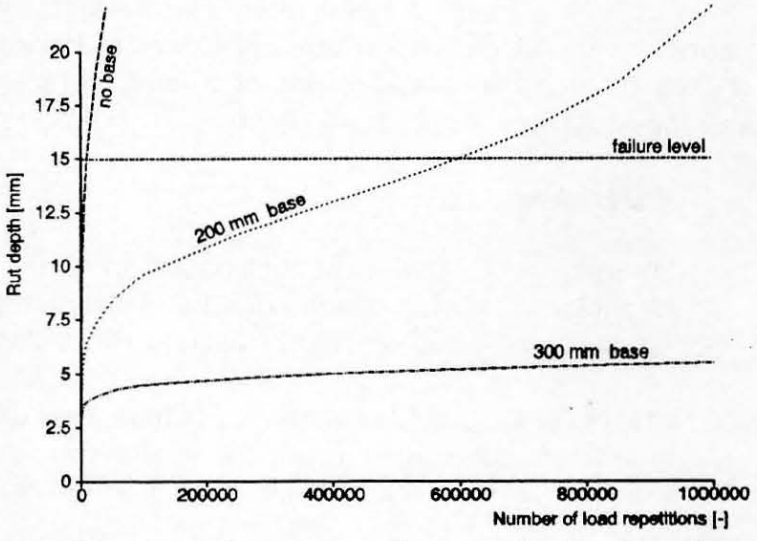


Fig. 12 Development of rut depth in three concrete block pavement structures.

4 Discussion

The research described in this paper contributes to the knowledge of concrete block pavement behaviour. Until now most of this knowledge is based on observations of real pavements, trafficked by real traffic.

Using the work presented in this paper it is possible to estimate the effects of new substructure designs, on the behaviour of a pavement, without having really construct it. The work presented furthermore makes it possible to understand how these effects are formed.

It is clear that the discussed calculations can be repeated. By varying layer thicknesses, layer materials, wheel load magnitudes, wheel load contact areas, the

standard deviation of lateral wander, and the stiffness of the subgrade a more general insight into the behaviour of concrete block pavements is obtained. This amongst others will give insight into the effects of various wheel loads on rut development. Vehicle simulation gives insight into the dynamic wheel loads, which of course are not constant over the chainage. Knowing the influence of various wheel loads on rut development, it is possible to determine the effects of the dynamic wheel loads of a vehicle on the initial longitudinal profile. An estimation of the behaviour of CBP concerning the development of longitudinal and transversal unevenness is now possible as is discussed in the paper "Structural Design of Concrete Block Pavements based on transversal and longitudinal unevenness" [5].

5. Acknowledgements

The following organisations are gratefully acknowledged for financially supporting this research into Concrete Block Pavement behaviour:

STW	Netherlands Technology Foundation,
VNC	Dutch Cement Association,
Febelcem	Belgium Cement Association,
Centre ROW	Centre for Research and Contract Standardization in Civil and Traffic Engineering - The Netherlands,
Fabes	Dutch Association of Concrete Paving Block Manufacturers,
NVWB	Dutch Association of Road Contractors

and the Municipal Works Rotterdam.

6. References

1. Huurman, M., "Resilient behaviour of Concrete Block Pavements and the Granular Materials used in the Substructure", Proceedings of the Fifth International Conference on Concrete Block Paving, Tel Aviv, Israel, 1996.

2. Francken, L. and Clauwaert, C., "Characterization and structural Assessment of bound materials for flexible road structures", Proceedings Sixth International Conference Structural Design of Asphalt Pavements, Ann Arbor, USA, 1987, pp. 130-144.

3. Veverka, V., "Estimation of the rut depth for roads with an asphaltic pavement" (in French and Dutch), La Technique Routiere, Volume XXIV, No. 3/1979, pp. 25-45.

4. Buiter, R., Cortenraad, W.M.H., Eck, A.C. van and Rij, H. van, "Effects of Transversal Distribution of Heavy Vehicles on Thickness Design of Full-Depth Asphalt Pavements", Transportation Research Record TRR 1227, Transportation Research Board TRB, Washington, DC, USA, 1989, pp. 66-74.

5. Huurman, M., "Structural Design of Concrete Block Pavements based on transversal and longitudinal unevenness", Proceedings of the Fifth International Conference on Concrete Block Paving, Tel Aviv, Israel, 1996.