SITE INVESTIGATIONS OF AMSTERDAM CONCRETE BLOCK PAVEMENTS

G.H. Kellersmann *, D.J. Nonneman *, E. Beuving **, A.A.A. Molenaar **.

* Road Department, Public Works, City of Amsterdam
** Netherlands Pavement Consultants

Summary

In 1980 the Road Department of Public Works of the city of Amsterdam decided to start a research program in order to study the behaviour of roads with concrete blocks.
At the Second International Conference on Concrete Block Paving the first results were published and a preliminary design method was presented. At that time some basic assumptions had to be made on the magnitude of the constants in permanent deformation law that was used, to describe the rutting behaviour of the block pavements.

After the conference in 1984, a new research program was started to derive the constants of the rutting law form observations of in situ pavements.
Test sections have been built with a base course, different jointing sands have been used and in two cases cementing material was added to the jointing sand.
Regular ruth depth measurements were made and also falling weight measurements were performed. In this way the permanent deformation as well as the lock-up phenomena, which is characteristic to block pavements, was studied and new constants for the rutting law were calculated.

In a test section in the Ookmeerweg, soil pressure measurements were made on the, in 1983 made, test section. These soil pressure measurements were done by using the falling weight deflectometer as loading device. Different load magnitudes were used and the stress dependent behaviour of the pavement was determined.
These soil pressure measurements are compared with the results of 1982 and 1983.
From the analysis of the measured results it appears that the vertical stresses in the bedding sand under the concrete block layer and the vertical stresses under the base course layer, can be calculated using a lineair elastic multilayer computerprogram, using E-moduli determined from falling weight deflections measurements.

Introduction

In the city of Amsterdam, about 50% of the roads consists of small element paving. This is more than 10 million m^2 pavement. To study the behaviour of these pavements, a research program was started in 1980 by the Road Department of the Public Works.

The aim of this research program was to set up a simple set of design curves for concrete block pavements that could be used by the Dutch municipal road authorities. In order to derive such design charts several experimental sections were installed.

The results of the experiments and the calculations were published at the Second International Conference on Concrete Block Paving, which was held in Delft in 1984 [1].

The design charts presented at that conference were based on certain assumptions on the constants in the adopted permanent deformation law. At that time it was recognized that further research was needed to refine and verify the design charts.
Therefore a new research program was started after the 1984 Conference.

In order to achieve the ultimate goal, which was the validation and refinement of the proposed design method, a test program was set up that can be devided into two parts
- tests on existing pavements and existing test sections.
- tests on new test sections.

This paper deals with two aspects of the mentioned research program. The first item is the soil pressure measurements with the existing load cells in the Ookmeerweg. These cells were also used in 1982 and 1983.
The tests were performed to investigate the behaviour of the 5 year old concrete block pavement. The latest results are compared with the earlier obtained ones to check if there are changes in time due to e.g. the lock-up effect.
The second item is the rutting prediction model used in the preliminary design curves. The rutting was calculated using the following permanent deformation law.

$$u_{pl} = u_{el} \cdot a \cdot N^b$$

where u_{pl} = permanent deformation due to N load repetitions
u_{el} = elastic deformation
a,b = constants

Since information on the magnitude of a and b wax absent, reasonable assumptions had to be made to develop the preliminary design charts. In order to obtain realistic values for a and b regular rut depth measurements were made. These measurements were performed on 6 sections. In thoss test sections also the effect of different joint filling material was studied.

Soil pressure measurements

In 1982 a total of 10 soil pressure cells has been installed at two different depths, in the test section in the Ookmeerweg (Figure 1 and 2).



Figure 1 : The begin of test section Ookmeerweg.

Five pressure cells were situated in the middle of the bedding sand layer between the concrete block layer and the base course layer. They are located at about 120 mm below road surface. Also 5 pressure cells were situated just below the base course, 350 mm below the road surface. The Kyowa cells were positioned in one line in driving direction. The distance between the cells is 150 mm. Figures 3 and 4 show the position of the cells in the road.

The first soil pressure measurements have been performed in October 1982, just after finishing the test section.



Figure 2 : Test section Ookmeerweg.

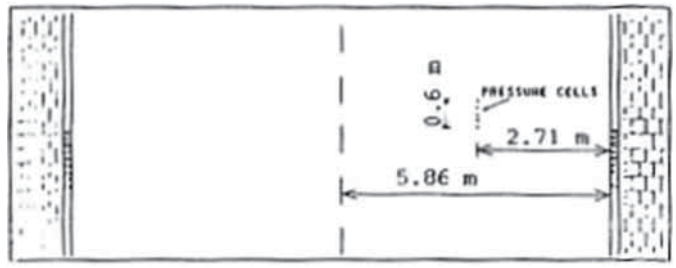


Figure 3 : Position of cells in the road.

Those soil pressure measurements consisted of:
- static and dynamic measurements with standing and driving trucks.
- dynamic measurements as a result of the pulse load of a falling weight deflectometer (F.W.D.).

The second series of measurements have been carried out in March 1983, which is half a year after construction of the test section was completed. The load used for these measurements consisted of a 50 kN impact load of a F.W.D. The results of these measurements have been reported in (1).

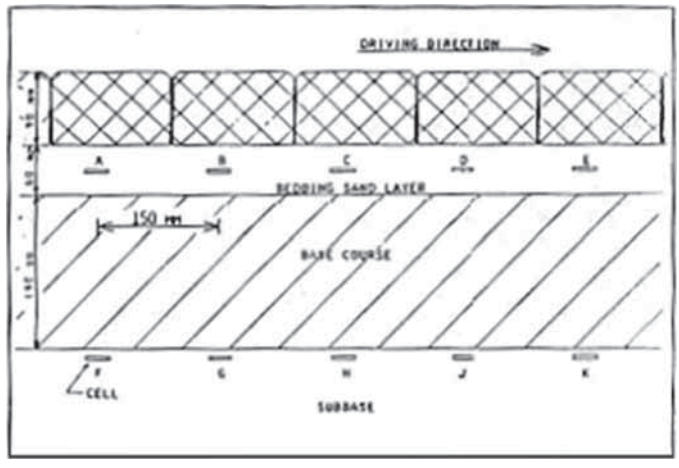


Figure 4 : Position of cells in the pavements.

Since eight of the ten loads cells were still working, 5 years after instalment, it was possible to perform a third series of soil pressure measurements in November 1987.
Again use was made of pulse loads caused by a falling weight deflectometer. The F.W.D. was used to be able to locate load pulses accurately at the marked positions. Besides that it was possible to vary the magnitude of the load impulse between 25 and 70 kN. The deflections at the pavement surface due to the load impulse could be measured at the same time.

The loads applied in 1987 were located in three lines.
One line was located over the load cells in the driving direction (centre line) and two lines were located parallel to the centre line at 150 mm distance to the left and to the right. Each series of measurements consisted of 31 locations over a length 2 m.
At each location 4 different load pulses were applied. The first load pulse was 70 kN, the

second load pulse about 50 kN, the third 35 kN and the last one 25 kN.
Using these four load levels it was possible to measure a stress dependent behaviour of the pavement if present.

The falling weight deflectometer (F.W.D.) used is shown in figure 5.



Figure 5 : Falling weight deflectometer.

The F.W.D. measures the deflections due to the pulse load in the load centre and at six other distances from the centre of the load. The selected pos
1.00, 1.50 and 2.00 m. The loads applied on the test section were 70, 50, 35 and 25 kN. The radius of the loading plate of the F.W.D. is 150 mm, so using a 50 kN pulse load, the uniformly distributed contact stress on the road surface is 0.707 MPa.

Some of the results of the soil pressures measured from loadings at the centre line are shown in figures 6 and 7. Figure 6 shows the curves of cell E, which is placed in the bedding sand layer.
The soil pressure curves of cell G measured under the base course are shown in figure 7.

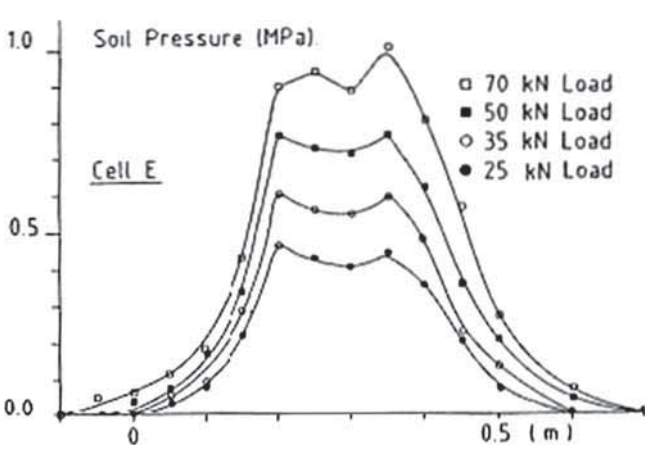


Figure 6 : Soil pressure curves, from 4 load levels at centre line. Cell in bedding sand layer.

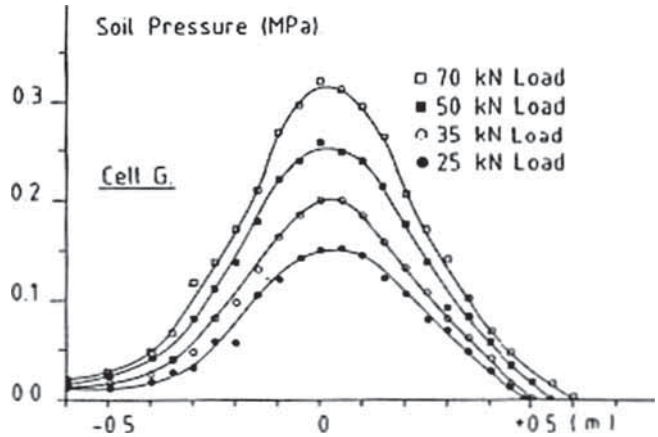


Figure 7 : Soil pressure curve under base course layer, from 4 load levels at centre line.

From figure 6 it appears that the vertical pressure in the bedding sand layer is around 0.7 MPa at a load level of 50 kN. This implies that the concrete block layer did not provide any load spreading under the pulse load of the falling weight deflectometer. Also one can observe that the increase in soil pressure in the bedding sand layer is almost the applied load.
Comparison of figure 7 with figure 6 shows that a considerable amount of stress reduction has occured due to the load spreading effect of the base. One will notice however that the ratio of stress in the bedding sand layer to stress underneath the base increases with increasing applied load level. This means that the base behaves non linear and reacts stiffer under higher loads.

The results of the measurements with loadings on the two lines parallel to the centre line at 0.15 m right and left are shown in figure 8 and 9.

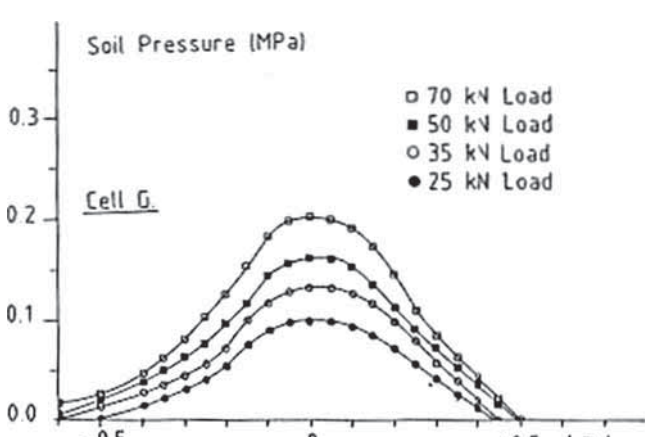


Figure 8 : Soil pressure curves from 4 load levels 0.15 m right from the centre line.

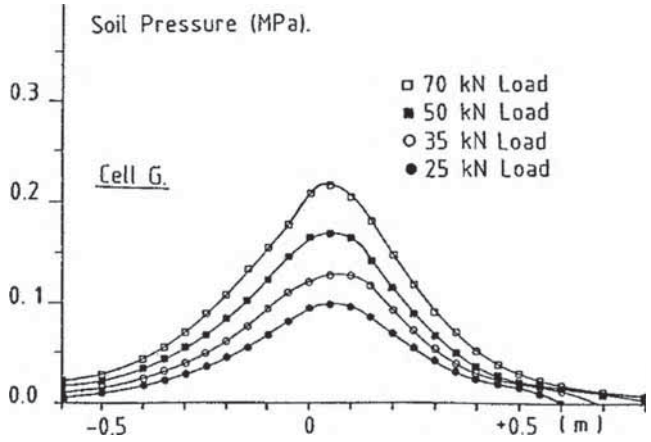


Figure 9 : Soil pressure curves from 4 load levels 0.15 m left from the centre line.

The stress distribution curves obtained for the upper cells in 1987 show a more smooth distribution than the curves obtained in 1983 (figure 10). The maximum soil pressure in the bedding sand layer (at a 50 kN load level) as measured in 1987 is lower than the maximum pressure measured in 1983.

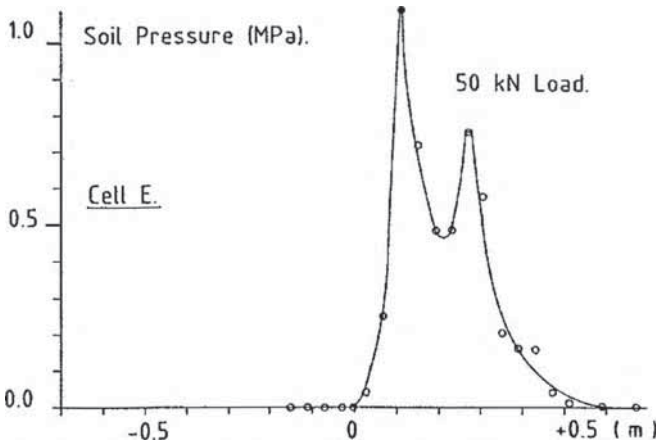


Figure 10 : Soil pressure curve in bedding sand layer (1983).

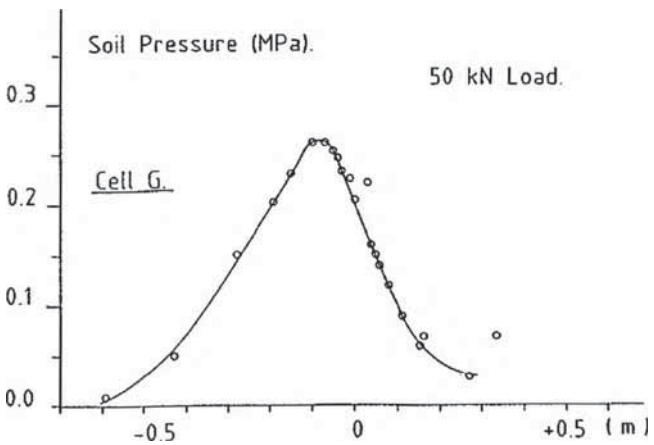


Figure 11 : Soil pressure curve under base course layer (1982).

The results of the lower cells that are located under the base course show that the same pressure level in 1987 as in 1982 (figure 11). However the 1987 results are a little bit higher than those obtained in 1983 (figure 12).

Stress dependent behaviour

As mentioned before, the soil pressure measurements were performed at four different load levels.

Figure 13 shows the relation between the ratio applied load ÷ 25 kN load and the ratio of measured soil pressure under the base course layer at the applied load ÷ the soil pressure due to the 25 kN load.
Figure 13 shows an increase of the soil pressure with 80% at a load increase of 100%. This means that the load spreading capability of the pavement increases with an increasing load and it indicates the stress dependent behaviour of the base.

This result was supported by the results of the F.W.D. measurements; since it was noticed that ratio of maximum deflection : applied load decreased with an increasing load level(figure 14).

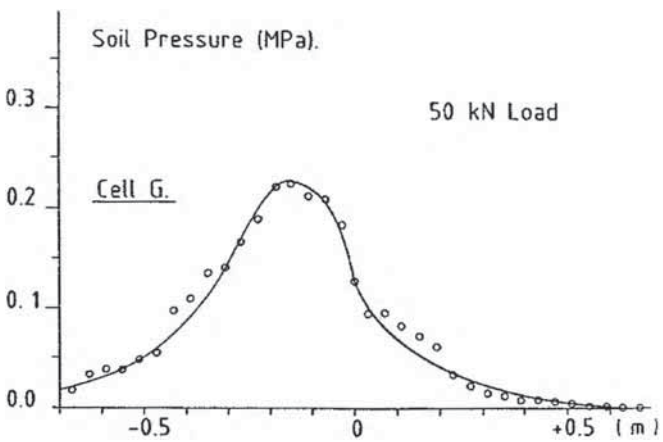


Figure 12 : Soil pressure curve under base course layer (1983).

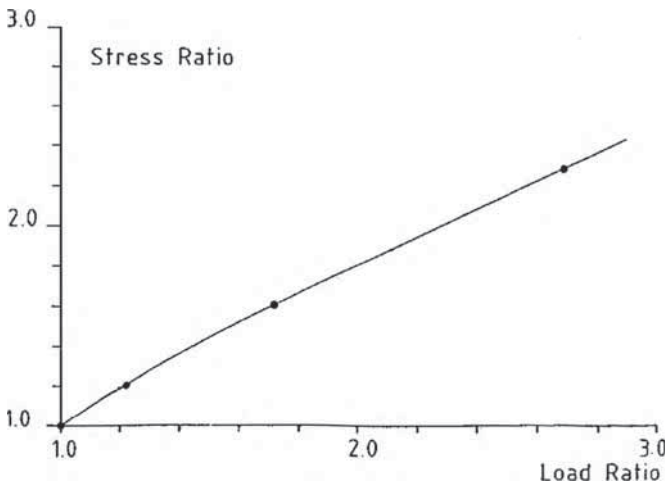


Figure 13 : Influence of load level on soil pressure under base course.

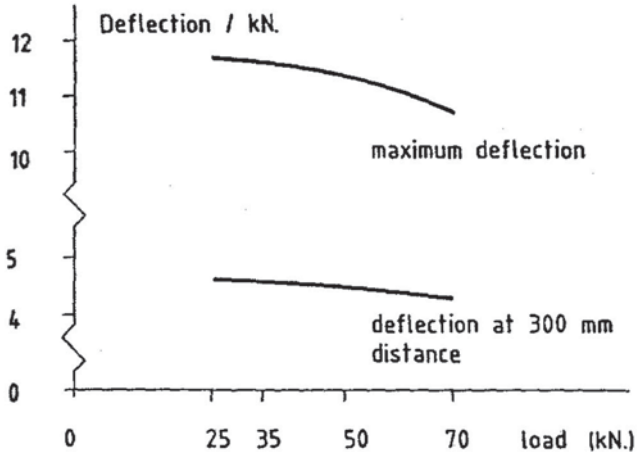


Figure 14 : Relation between measured deflection and F.W.D. load.

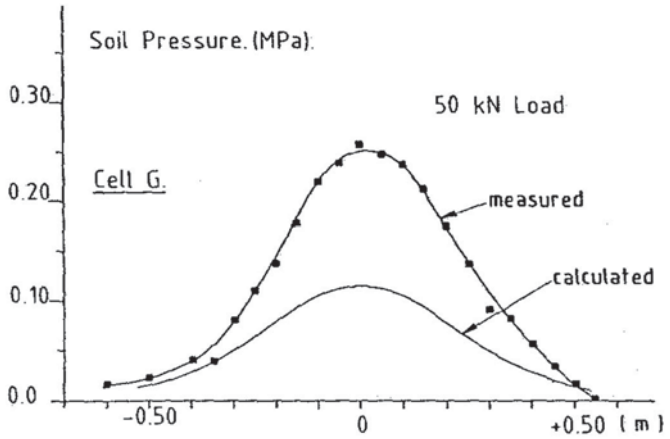


Figure 15 : Comparing measured stressed with calculated in model with concrete block layer.

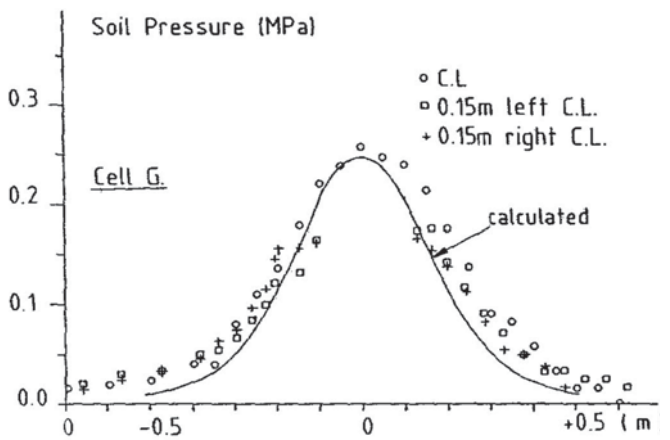


Figure 16 : Comparing measured stresses with calculated in pavement model without concrete block layer.

Measurements compared with calculations

In order to check whether or not the measured soil pressures could be explained by means of a structural analysis, knowledge on the elastic modulus of the different pavement layers should be obtained. This information was obtained by back calculating the E-moduli from the measured deflection profiles and the known layer thicknesses. A lineair elastic multi layer computer-program is used (Chevron) to simulate measured deflection profiles. The back calculated E-values are shown in table 1.

layer	thickness in mm	E-modul in MPa
concrete blocks	90	2500
bedding sand	60	100
base course	190	250
sand layer	1000	200
subgrade		135

table 1 : Back-calculated E-moduli

After the back calculation of the E-moduli the soil pressures due to the F.W.D. load under the base course were calculated. The calculated stresses were about half the measured stresses (f igure 15).

If the calculations were repeated but now with ommission of the concrete block layer, so the load is applied directly on the bedding sand layer, the calculated soil pressures are almost the same as the measured one. The comparison of the calculated stresses and the stresses measured at cell G is shown in figure 16.
The results indicate that with regard to the displacement behaviour, the behaviour of a concrete block pavement can be modelled as a linear elastic layer system. With regard to the stress distributions in the structure, the pavement can be modelled as a linear elastic layered systems but the block layer should be ommitted.
Such a contradictory behaviour could of course be expected since the concrete block layer obtains its strength purely of the shear resistance between the blocks and rotational friction. The block layer cannot cope with bending moments.
One could of course wonder whether or not this conclusion is influenced by the type of loading that was used in this test program. The pulse load of the F.W.D. is generated on only a limited number of blocks and one could state that only the shear resistance between the blocks is activated. If this is true, significant lower soil pressure should be measured using a rolling wheel load of the same magnitude. In the previous study however it was shown that this is not the case (figure 17). The mesured soil pressures due to a rolling wheel load were the same as those measured under a pulse load of the same magnitude.

This, leads to the following conclusions:
a. the surface deflections of a concrete block pavement can be calculated by means of a linear elastic multi-layer model.

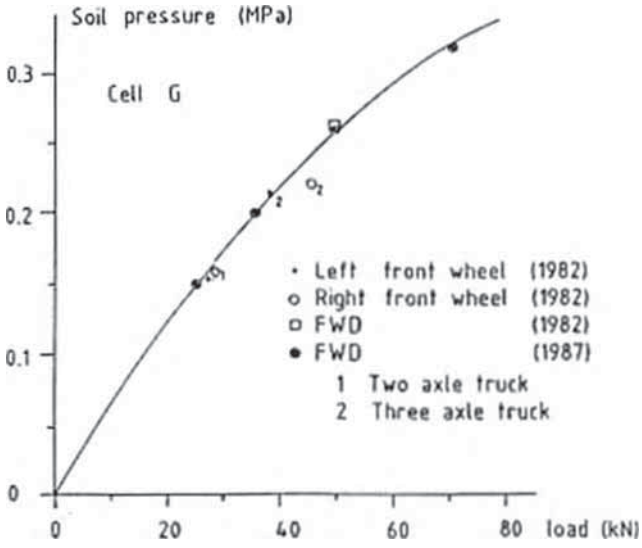


Figure 17 : Relation between soil pressure and single wheel Load and F.W.D. load.

b. the vertical pressures in a pavement with a concrete block top layer can be calculated by means of a linear elastic multi-layer model as well but the thickness of the block Layer should be ommitted in such an analysis.

Verification of the ruth depth prediction model

Permanent deformation (rutting) is the most important structural defect of concrete block pavements. The design of these pavements should be such that the maximum alowable rut depth is not exceeded before the predefined design number of load repetitions has been reached. In (1) the ruth depth development has been calculated using the equation.

$$u_p = u_{el} \cdot a \cdot N^b$$

where

u_p = permanent deformation (mm)

u_{el} = elastic deformation (mm)

N = number of load repetitions that cause an elastic deformation of u_{el}

a,b = constants

This rut depth equation has been developed by researchers of the Belgium Road Research Centre (2).
The original equation had been developed for different types of material. In this case however the permanent deformation of the overall construction is calculated directly from the elastic deformation of the overall construction (maximum deflection at a 50 kN load) and not from the permanent deformation of the individual layers. Tils means that the constants a and b should be related to the behaviour of the overall construction.

The selection of proper values for the constants a and b is very important. In the previous study (1) estimates have been made for a and b in relation to the materials used in concrete block pavements. However these estimates should be validated by means of an evaluation of in situ pavement sections. Therefore rut depth measurements were performed on test sections with concrete block pavements on the Singel in Amsterdam.

Table 2 shows the different test sections in the Singel.

Test section
Singel 1,2,3 and 4: 90mm concrete blocks (100 x 200 mm) 70mm bedding sand 200mm crushed old concrete
Singel 1 : normal jointing snad between concrete blocks 2 : crushed jointing sand between concrete blocks 3 : normal jointing sand with 2% cementing material 4 : crushed jointing sand with 2% cementing material
Singel B : 95mm concrete blocks (100 x 200 mm) herringbone 50-100 mm bedding sand laid 200 mm crushed old concrete 0/4 mm

table 2 : Construction of test sections

The test sections Singel 1, 2, 3 and 4 were made in January 1984 (figure 16).
Test section Singel S was built in November 1984.



Figure 18 : Applying jointing sand on test sections Singel.

In the period between 1984 and 1986 regular rut depth measurements were performed. Also F.W.D. measurements were carried out frequently.
Besides measurements on the Singel, also measurements were performed on the already mentioned test section of the Ookmeerweg.

In every section the rut depth measurements were performed in both wheel tracks and at 10 locations each.
All Singel sections show more rutting in the right hand wheel track (that is the track closest to the pavement edge) than in the left hand wheel track (that is the track closest to the adjacent canal). Such a behaviour was not observed at the Ookmeerweg. Table 3 shows the results at the end of the test period.

Section	age (month)	rut depth (in mm)	
		left track	right track
Singel 1	34	2.0	10.0
2	34	3.2	10.8
3	34	4.0	14.0
4	34	8.2	15.3
Singel B	25	2.0	5.7
Ookmeerweg	49	9.6	2.7

table 3 : Rutting at the end of the test p

Figure 19 shows the development of the rut depth with time of Singel 3.

Besides the ruth depth measurements also F.W.D. measurements were performed. Figure 20 shows as example the results of the F.W.D. measurements on section Singel 3,left wheeltrack and figure 21 shows the measured deflections at Singel B right wheel track.

In figure 20 a variation with time is shown for the deflections 0, 200 and 300 mm. Figure 21 shows a relative rapid decrease of the measured maximum deflection during the first half year after construction. After that the maximum deflection seems to vary around a mean value of about 700 pm. This variation is most probably due to sensorial variations and other climatic related influences. All in all it means that the stiffness of the pavement increases rapidly with time especially in its early life.

Back calculations of E-moduli from the F.W.D. deflection profiles show an increase of the E-moduli of the concrete block layer from about 750 MPa (1984) to about 2200 MPa (1986). This increase of E-moduli is the lock-up effect. No significant influence was found from the jointsand type on the E-modulus of the concrete block layer.
The E-moduli of the bedding sand layer show in many cases a large fluctuation between 30 - 125 MPa. This fluctuation is attributed to climatic influences.

Determination of a and b values

Based on the ruth depth measurements, the measured deflections and traffic data expressed in 50 kN equivalent wheel loads, the constants a and b of equation (1) were calculated.

In this equation u_{e1} was determined by F.W.D. measurements with a 50 kN pulse load.
The results of this analysis are shown in table 4.

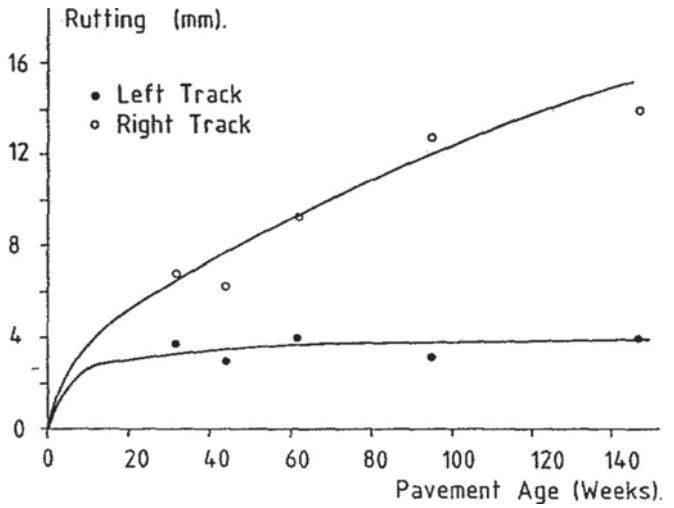


Figure 19 : Increase of rut depth with time. Singel 3.

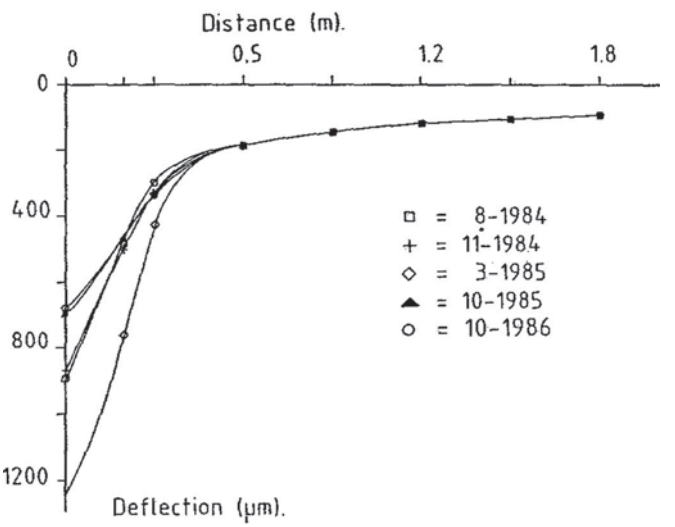


Figure 20 : F.W.D. results Singel 3 left wheel track.

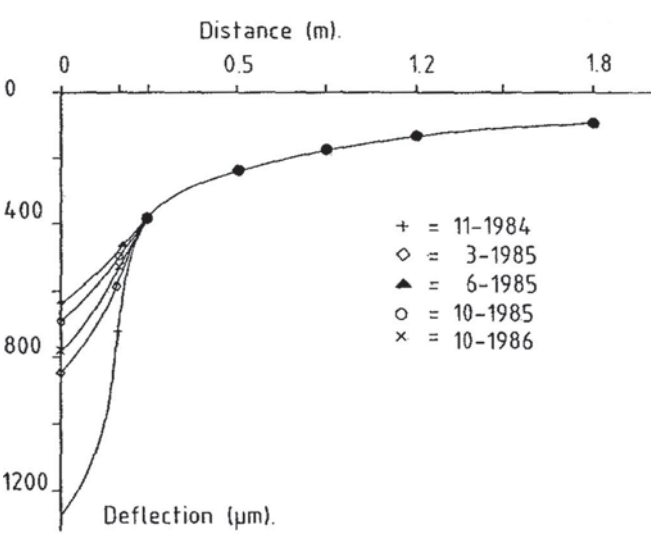


Figure 21 : F.W.D. results Singel B right wheel track.

Section	wheel track			
	left		right	
	a	b	a	b
Singel 1	0.12	0.40	0.50	0.59
Singel 2	0.29	0.25	0.08	0.55
Singel 3	1.10	0.15	0.19	0.50
Singel 4	0.30	0.50	0.21	0.50
Singel B	0.18	0.25	0.11	0.40
Ookmeerweg	0.26	0.40	0.05	0.45
Mean value	0.33	0.33	0.19	0.50

table 4 : Calculated a and b values.

The results in table 4 are compared with the values for a and b as reported in three different studies. This comparison is given in table 5.
With regard to the a and b values that were found for the different sections, the following comments can be made. The sections Singel 1-4 have a width verying between 3 and 3.4 m. The Singel B section has a width of 3.5 m. This means that on these narrow one lane pavements almost all trucks will drive in the same wheelpath and the edge constraint is rather limited. On the 11.72 m two lane wide Ookmeerweg the trucks have much more freedom to chose their own path and much more lateral wander will occur than on the Singel sections. This might have a significant influence on the obtained a and b values.

In order to know whether use of the original a and b values in the rut depth model results in significant different rut depth prediction than if the a and b values are used resulting from this research program, a comparison was made between the observed and predicted behaviour for section Singel 3. This comparison is shown in figure 22. From this figure one will observe that the rut depth predictions are in pretty good agreement with the observed out depth in the right hand track. For the left hand track however a significant difference in observed and predicted behaviour can be observed.

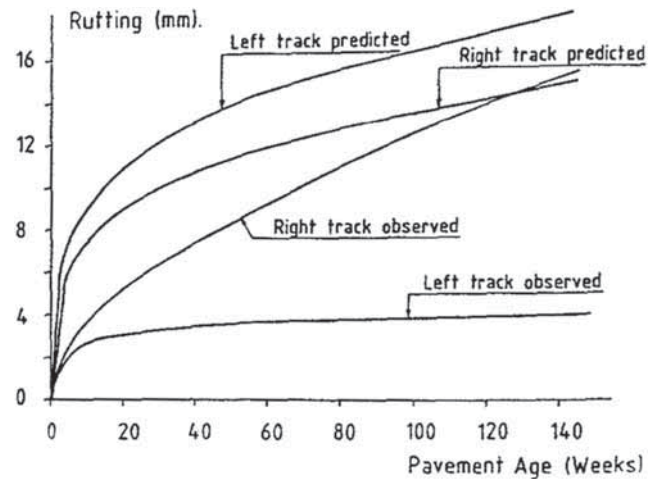


Figure 22 : Comparison between the observed and predicted (a-2 and b=0.25) behaviour of section Singel 3.

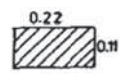
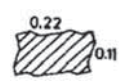
Material/ Construction Type Type of loading	Ref.	Bond Type	Block Type	Thicknt;ss [m] Block	Base	Stiffness Subgrade [MPa]	a	b
Sand subbase dynamic triaxial	2						2.0	0.30
Granular subbase dynamic triaxial	2						2.0	0.20
Cone. Block Pav. Rep.plate loading	3	Herring bone 45°		0.08	-	137	0.48	0.38
				0.12	-	144	1.73	0.22
Cone.Block Pav. Load simulator	4	Herring bone 45°		0.10	0.16	CBR=66%	0.22	0.22
				0.08	0.10	id.	0.26	0.23
				0.06	0.06	id.	0.24	0.34

table 5 : Values for a and b in relation to Material and Construction Type

Conclusions

Based on the results of the research program reported in this paper the following conclusions can be drawn.

1. Concrete block pavements can be analyzed by means of a linear elastic multi layer analysis, but the following aspects should be taken into account.
   - for a stress analysis of the base, subbase and subgrade, the concrete block layer should be ommitted otherwise the predicted stressed are too low.
   - for a surface deflection analysis, the entire structure should be taken into account which means inclusive of the concrete block layer.

2. Rut depth predictions can be properly made using the maximum deflection as input. This implies that deflection measurements are recommended to be performed directly after construction is completed in order to determine the rut depth potential of the pavement.

3. For design purposes a value of *0.2* for a and a value of 0.3 for b are recommended to be used in the rutting prediction model.

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