THE APPLICATION AND VALIDATION OF AN ANALYTICAL METHOD IN THE STRUCTURAL REHABILITATION OF A CRACKED RIGID PAVEMENT

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The use of high alkali cement together with aggregates rich in silica to construct a jointed unreinforced concrete pavement without dowels resulted in a chemical reaction that caused microscopic cracking. This cracking not only reduced the stiffness of the slab but also reduced the strength of the subbase so that structural cracking developed under traffic loading. The visual assessment of the structural condition of the pavement was based on a load transfer constant which in turn correlated with deflections and relative vertical movements at the joints. In order to relate the visual condition to pavement response and eventually performance, an analytical model was used to calculate pavement response in terms of vertical movement and deflections. Experimental overlay sections, using Portland cement concrete, continuously graded crushed stone and asphaltic concrete, were constructed to calibrate the analytical model and these sections are being trafficked by road users in addition to a mobile accelerated traffic simulator, the Heavy Vehicle Simulator, to evaluate the relative performance of each section. This paper discusses the findings on some of the concrete overlay sections and the data is extrapolated to include preliminary predictions of the performance of those overlays that have not yet been tested by the HVS.

The chemical reaction between high alkali cement and siliceous gravel initiates cracks in concrete and, if it occurs in a concrete slab, it can reduce slab stiffness and increase deflection under loads and therefore increase the stress on the subbase and subgrade. Since overstressing of the sublayers causes deterioration of these layers and eventually also the concrete layer, structural cracking is inevitable. An increase in permeability of the slab due to cracking allows surface water to enter the pavement and abrasion of the pavement structure rapidly develops into structural failures.

Overlays are some of the most common rehabilitation measures for a distressed pavement and, in order to design such an overlay properly, an evaluation of the existing pavement followed by appropriate design techniques is necessary. The performance of such an overlay can to a large extent be predicted by means of computer and laboratory techniques. However, environmental factors greatly influence performance and should therefore be taken into consideration in the design and construction of overlays.

A jointed unreinforced concrete pavement (sometimes known as a plain concrete pavement) in South Africa has cracked owing to this alkali-aggregate reaction and subsequently traffic loading has aggrevated its structural condition. In order to plan rehabilitation, the pavement was evaluated by correlating a low-cost visual condition survey with more costly non-destructive testing of pavement response. By implementing stochastics as well as finite element techniques, an analytical model was developed by which the pavement response could be simulated. Non-linear crack and joint behaviour had to be taken into account to obtain a fairly high accuracy of simulation. This analytical model was used for comparing different rehabilitation alternatives such as undersealing, "freezing" of joints and different types and thicknesses of overlays. Furthermore overlays were designed and eventually built as experimental sections for the purpose of field validation and calibration of the analytical model.

This paper discusses the design, construction and selection of the most appropriate experimental overlays, which include various sections of Portland cement concrete, asphaltic concrete and continuously graded crushed stone. These sections were subjected to accelerated traffic loading by the Heavy Vehicle Simulator (HVS) in order to speed up distress so that a final decision on the most appropriate action could be made.

Deterioration of the slab

The deterioration of the pavement structure was initiated by a chemical reaction which caused small cracks. These cracks developed into structural cracks under traffic loads and eventually spalls and punch-outs occurred.

Chemical reaction and cracking

A variety of reactions are known to occur between cement and aggregate (1). Some of these reactions are deleterious, causing disruptive expansion, whilst others are beneficial, for example the pozzolanic effect responsible for the strengthening of the interfacial bonds between some siliceous aggregates and the surrounding matrix of cement paste. Alkali-aggregate reaction, as referred to in this paper, implies a chemical reaction between a cement high in alkali content and reactive constituents such as SiO_2 in the aggregate. This reaction can be detected visually and shows up as a rim around the periphery of the coarse aggregate. The reaction is expansive by nature and causes cracking of the structure as shown in Figure 1. Initial cracks are fine with a trace of discoloration on the surface of the slab, but cracking can obviously occur wherever alkali-aggregate reaction takes place. It was found for instance that horizontal cracking within the slab as well as longitudinal cracking in the road pavement also occurred, as can be seen in Figure 1.

Structural cracking and pavement condition

Cracking in any concrete slab is indicative of a reduction in slab stiffness. In this particular case cracking from the surface downwards not only reduces the effective modulus of stiffness of the concrete but also the effective thickness of the slab. The result of the decrease in slab stiffness is an appreciable increase in deflection, which can be illustrated by the equation:

$$y = C \frac{P}{\sqrt{Dk}} \tag{1}$$

where y = deflection
C = a constant that depends on the position of the load on the slab, slab geometry, etc.
P = load on slab
k = slab support
D = slab stiffness

$$\text{and } D = \frac{Eh^3}{12(1 - \mu^2)} \tag{2}$$

E = modulus of stiffness of the concrete
h = slab thickness
μ = Poisson's ratio

The value of C depends on the position of the load on the slab and has a value of around 0.12 for interior loading, 0.45 for edge loading and 0.70 for corner loading of a pavement slab of average thickness and strength. If one accepts that the joint in a slab has been perfectly constructed with full load transfer from one slab to the other, a C value of 0.23 can be assigned for this joint loading condition. This distinct difference between the values of C has been used as a standard to be applied in a condition survey, and Table 1 gives an indication of the visual condition of the pavement as related to the condition rating or load transfer constant C. Figure 2 indicates a C value of 0.50.

Table 1. Condition survey as related to the load transfer constant C.

Condition of Slab	C
Perfect joint	0.23
Fine cracks next to joint	0.35
Well-defined cracks	0.50
Spalling and cracks with pumping	0.60
Punch-out, patches	0.70

It is clear from Table 1 that an increase in the deflection of a concrete pavement is expected as the joint deteriorates. Actual deflections were taken using a Benkelman beam and plate loading equipment as well as Multi-Depth Deflectometers (MDD) used with the Heavy Vehicle Simulator (HVS) (2). The MDD is used to measure deflections at various depths in the pavement structure whilst the pavement is being loaded. This will be discussed in more detail later on in the paper. Figure 3 shows a plot of deflection measurements against pavement condition in terms of the C value as defined in Table 1.

Relative vertical movements were also measured at the joints by using a modified Benkelman beam and plate load equipment supplemented with MDD readings (3). Figure 4 shows a plot for a static loaded condition such as the plate load test and Figure 5 the results when a load moves across the joint from one slab to the other. It is clear that loading on one slab only, the static case in Figure 4, resulted in about half the relative vertical movements as were measured under a moving load, Figure 5. Measurement of horizontal movement due to temperature changes indicated a small coefficient of expansion. This is attributed to the fact that the whole pavement is in compression owing to the alkali-aggregate reaction. Thermal expansion therefore occurs vertically and transversely, as also indicated by horizontal cracks within the slab as well as longitudinal cracks on the surface.

The deterioration of the concrete pavement can therefore be summarized as shown in Figure 6. Cracks initiate in the concrete owing to a chemical reaction between alkaline cement, aggregate high in silica content and environmental influences such as water. The further development of these cracks into structural cracking can be prevented by maintenance but in practice this has been proved to be expensive and not very effective. The smaller cracks initiated by alkali-aggregate reaction reduce the slab stiffness and increase deflection as well as relative movement at the joints. This results in higher stresses on the subbase, which also deteriorates owing to the alkali-aggregate reaction, since the same cement and aggregate were used in the construction of this layer. Traffic loading therefore causes higher stresses in both the concrete slab and the subbase, and with water penetrating the joints structural cracking is inevitable. In time the joints, the subbase and the slab itself deteriorate further and spalling as well as pumping results. At this time maintenance is of great importance to prevent structural failures, but where deterioration is allowed to progress a loss of subgrade support will lead to structural failures which can only be remedied by major rehabilitation. This final condition of the pavement is shown in Figure 7 where an attempt has been made to seal the cracks and to replace a punch-out with asphaltic concrete. Pumping at the cracks is also noticeable, with fine reaction cracks visible in the inner lane joint.

Figure 1. Cracking due to alkali aggregate reaction in a rigid pavement.

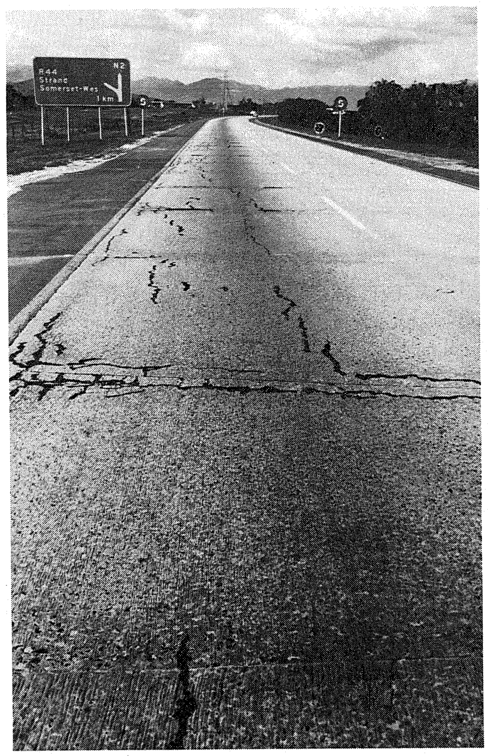


Figure 2. Development of small reaction cracks into load associated cracks (joint condition C = 0,5).

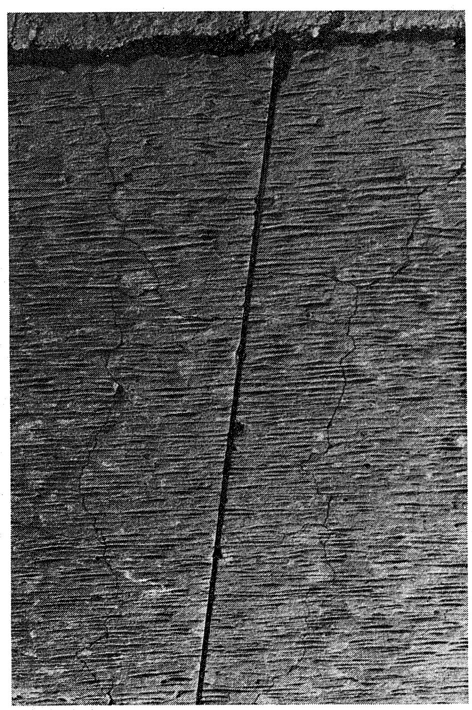


Figure 3. Deflection under a 40 kN load as a function of joint condition C.

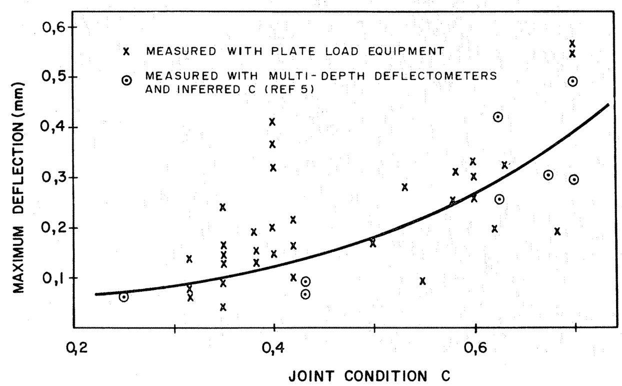


Figure 4. Relative movement at a joint under A 40 kN plate load as a function of joint condition.

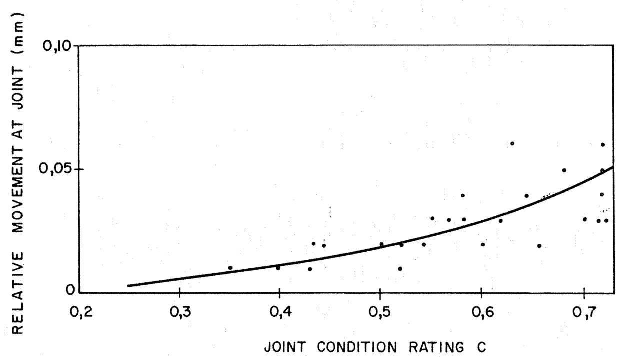


Figure 5. Relative movement at a joint under a 40 kN moving load as a function of joint condition C (measured by Benkelman Beam).

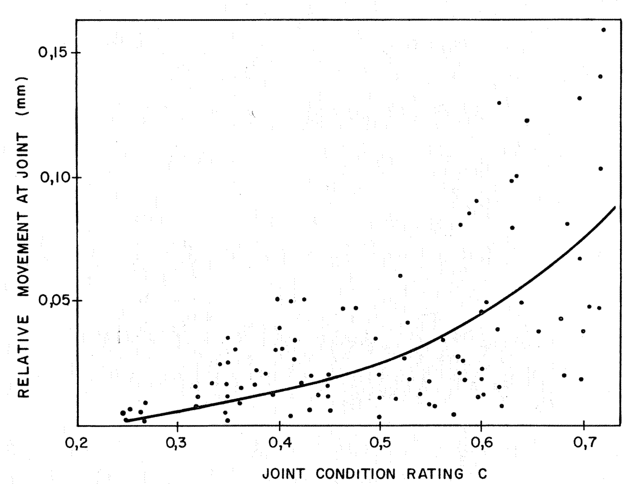


Figure 6. Factors leading to structural failure and appropriate action.

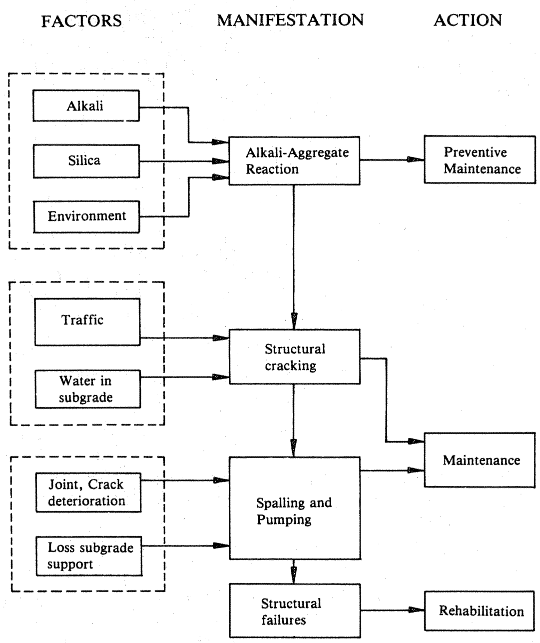
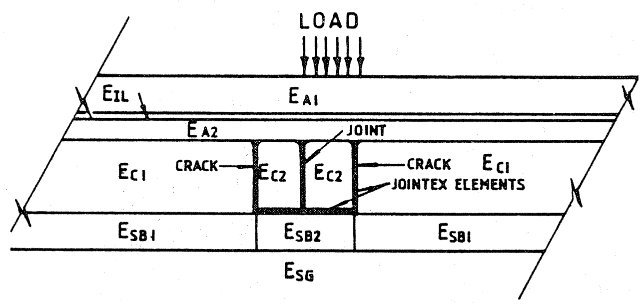


Figure 7. A failed joint with spalling, pumping and a punch out visible (condition rating 0,7).

Figure 8. Schematic view of analytic model.



Analytical modelling

The pavement at this stage deteriorated to such an extent that normal routine maintenance became ineffective and different rehabilitation strategies had to be considered. It became quite clear that overlaying is a viable solution but that reflection cracking must be avoided.

In order to evaluate the different options, a three-dimensional finite element model was constructed. This model, schematically shown in Figure 8, has the capabilities to handle both non-elastic and elasto-plastic materials, and it was used to simulate existing pavement response as depicted by deflections and relative vertical movements at the joints. The joints themselves were simulated using elements that could vary in tensile, compression and shear stiffness.

Table 2 shows a summary of the stiffness values of the different pavement components that had to be used in order to simulate the field measured deflection and relative vertical movement of a particular joint. Initially modulus values for the concrete were measured in both the laboratory and the field, and subgrade moduli were inferred from miniature plate load penetrometer and CBR tests (3). Unfortunately no acceptable subbase samples could be extracted for proper testing, and crude field tests had to be employed as a first step to obtain an indication.

The values in Table 2 show that it was the subbase that apparently experienced the most significant deterioration in stiffness. The joints and cracks deteriorated in their ability to transfer shear loads, owing to a decrease in aggregate interlock. The loss in subgrade support may be associated with a poor subgrade at these various locations, but it is more likely that voids existed below the slab at that stage which might be simulated in the analytical model either by a low subgrade stiffness, by an actual void, or by using the same element that was used to simulate a crack. However, for the purpose of this paper, an equivalent subgrade stiffness value is included in Table 2 for comparison.

Since deflection is normally associated with subgrade stiffness, a plot can be made to illustrate the relative sensitivity of deflection to subgrade and subbase stiffness, shown in Figure 9. A plot of relative vertical movement against subbase stiffness with subgrade stiffness as a third variable (Figure 10) shows that relative movement is more sensitive to subbase than subgrade stiffness.

Table 2. Simulated deteriation of the pavement.

Condition C	Deflection (mm)	Relative movement (mm)	Relative Stiffness (MPa)		
			Concrete	Subbase	Subgrade
0.3	0.07	0.008	28 000	8 000	400
0.5	0.14	0.025	21 000	1 200	300
0.7	0.38	0.075	18 500	500	60

Figure 9. Calculated deflections of the pavement under a 40 kN load as a function of subgrade and subbase stiffness.

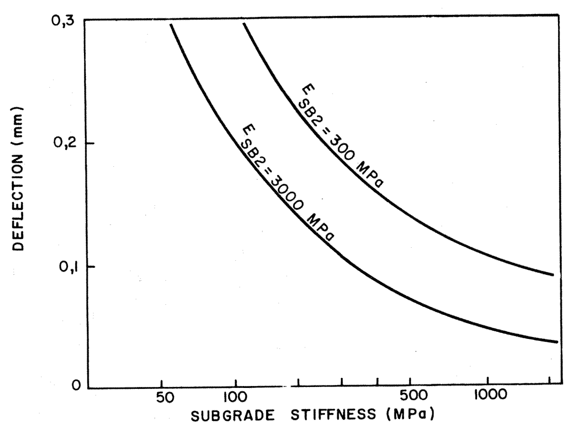
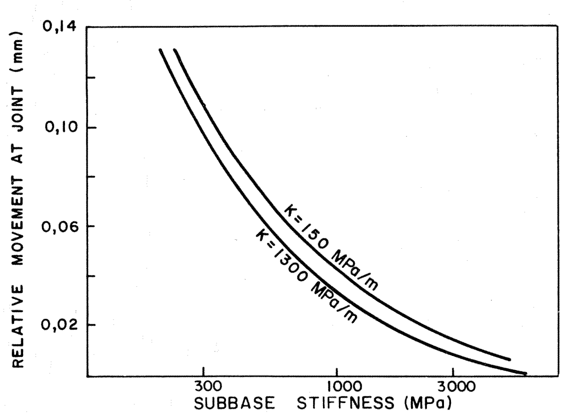


Figure 10. Relative movement at the joint under a 40 kN load as a function of subbase and subgrade stiffness.



The conclusion to be reached from the above modelling is that cracking in the slab increases stress on the subbase and subgrade but, with an increase in the cumulative number of load applications, cracks in the slab widen, stiffness of the slab decreases further and the subbase is gradually destroyed. The presence of water in the pavement structure causes pumping and a decrease in subgrade stiffness with a strong possibility of voids. Investigations of the pavement structure at various stages of deterioration indicated that at a condition rating C of 0.3 an intact sample of the subbase could be extracted, but this was not possible for a joint with a condition rating greater than 0.4.

Design of experimental rehabilitation

The analytical model was used to determine the relative sensitivity of different maintenance or rehabilitation options, to simulate theoretically the effect of stiffening the subbase or cementing the cracks in the slab. It was found that any of these two approaches individually would have a limited effect since the whole pavement structure had to be stiffened to avoid overstressing of the one component that had been renovated. This implied that the cracks in the slab would have to be cemented to increase slab stiffness, in addition to the stiffening up of the subbase. From a practical point of view this meant the undersealing of the slab as well as the grouting of the cracks in the slab. Both these measures were tried on a small scale and both proved ineffective when measured in terms of a decrease in deflection and relative vertical movement at the joints.

Various alternative rehabilitation options then had to be considered namely reconstruction, recycling or the overlaying of the pavement with different materials. The first two options could be evaluated by the use of basic principles, and certain assumptions regarding road users' costs and inconvenience could be made in order to evaluate the cost-effectiveness of these options. However, to evaluate the different overlays it was felt that more sophisticated techniques should be employed. The use of the analytical model in this case was of great significance, and different types of material with different thicknesses were superimposed on the model of the existing structure.

Three types of material were considered for use in overlays, namely Portland cement (PC) concrete, continuously graded crushed stone with an asphaltic wearing course, and asphaltic materials. The PC concrete options could be divided into unreinforced jointed or reinforced concrete overlays. Several reinforcement options were possible i.e. continuously, fibre or jointed reinforced as well as post-stressed overlays. The two options of plain jointed and continuously reinforced concrete were used for analysis.

The continuously graded stone option was included because this material, compacted to 86 % of absolute maximum density and with a stiffness modulus of about 500 MPa, has given excellent performance under heavy traffic in South Africa.

Many forms of asphaltic overlays were possible; a full-depth asphalt, thinner asphaltic concrete with different interlayers, and thin asphaltic concrete with asphalt rubber as the binder were the most usual options. Interlayers which could be considered included asphalt rubber membranes, woven and non-woven geofabrics, and other bituminous materials.

In order to design these different experimental overlays, the analytical model was used to calculate values for strains and displacements, as well as for normal and shear stresses in a three-dimensional medium. These response values could be used to relate to a fatigue curve of cumulative loads to failure. Unfortunately these fatigue curves were not available for all materials and loading conditions, especially if they were to be applied in designing overlays where reflection cracking was of concern. Thus the actual expected lives of the different overlays were very difficult to determine theoretically.

In order to relate theoretical analysis to actual field experience, the building of experimental sections and the monitoring of performance with time were essential. Since it was possible to subject the experimental sections to accelerated traffic with the Heavy Vehicle Simulator (HVS), a quick estimate of real-life performance under traffic loading was possible. The details of this accelerated loading program of the HVS, with some results, will be discussed in the next section.

The following experimental overlay sections were built with the layout shown in Figure 11 (4):

1. Continuously reinforced concrete pavement 100 and 125 mm thick with 0.67 per cent longitudinal reinforcement.
2. Jointed unreinforced concrete pavement, 125 and 150 mm thick with the joints spaced at 4.0 m.
3. Continuously graded crushed stone overlay, 150 and 200 mm thick, with a 40 mm semi-gap-graded asphaltic concrete wearing course with 19 mm rolled in chips.
4. Semi-gap-graded asphaltic concrete, 70 mm thick, with rolled-in chips over interlayers such as a single seal with 150/200 pen asphalt binder, asphalt rubber binder and a woven and non-woven geofabric.
5. Semi-gap-graded asphaltic concrete overlay ranging from 200 mm to 40 mm in thickness.
6. A 40 mm open-graded asphaltic concrete with asphalt rubber as a binder.

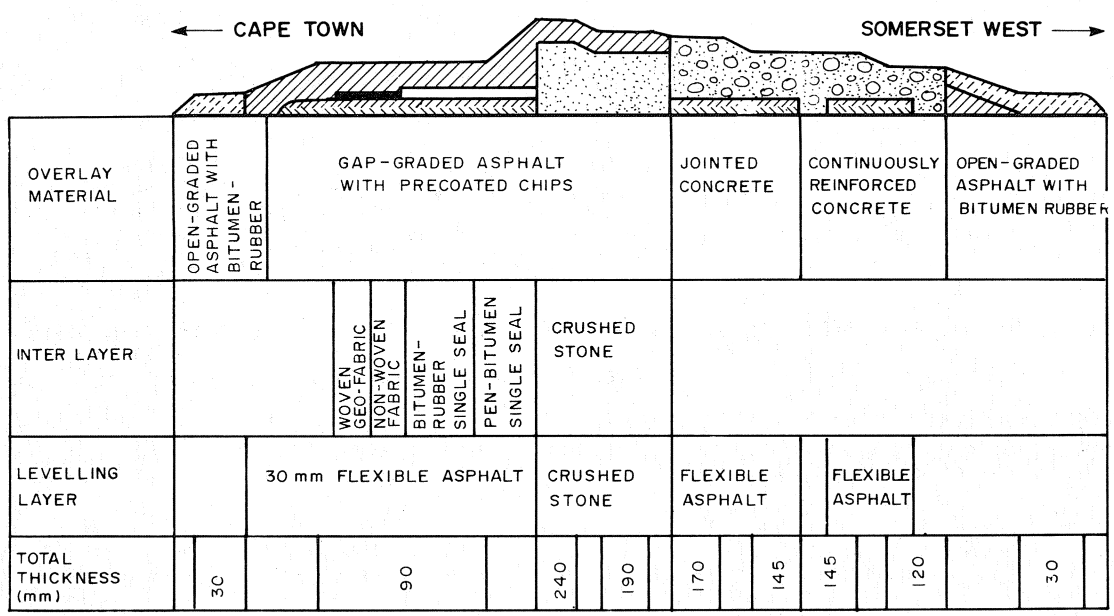


Figure 11. Basic layout of experimental sections.

Bonded sections were considered, but because of the occurrence of cracking and punch-outs, it was decided to stick to unbonded overlays. To ensure unbonded PC concrete overlays, a 25 mm flexible asphaltic concrete was placed on top of the old pavement. This layer served a double purpose: it acted as a bond breaker and reduced the risk of voids underneath the overlay in the event of warping. Joints in the existing pavement were at a 4.5 m spacing, and in order further to reduce the risk of warping, the joints in the jointed overlay were placed every 4.0 m.

Field validation

The experimental sections, covering a total length of 660 m, were completed at the beginning of 1983. The most inexpensive section was the thin layer of asphaltic concrete with asphalt rubber as a binder, while the Portland cement and the full depth asphalt overlays were the most expensive. To date all sections have performed very well. The 90 mm gap-graded asphalt concrete overlay has cracked at the old joints but the thin asphaltic concrete with asphalt rubber binder is behaving exceptionally well considering its thickness.

Since this thin asphaltic layer seems to be a very cost-effective solution and since it can also be regarded as the first phase of a stage construction solution for any of the other options, it was tested first under accelerated loading with the HVS.

Heavy vehicle simulator testing

The HVS (2), (Figure 12) is a mobile testing machine with which pavements can be subjected to accelerated traffic loading. The HVS test consists of the repeated application of a rolling wheel load to a pavement structure and several parameters can be varied (5) namely the magnitude of wheel loads, number of load repetitions and the

Figure 12. The heavy vehicle simulator.

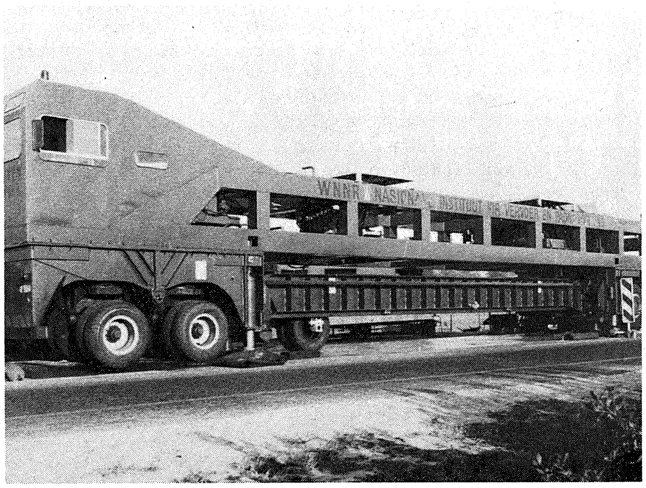
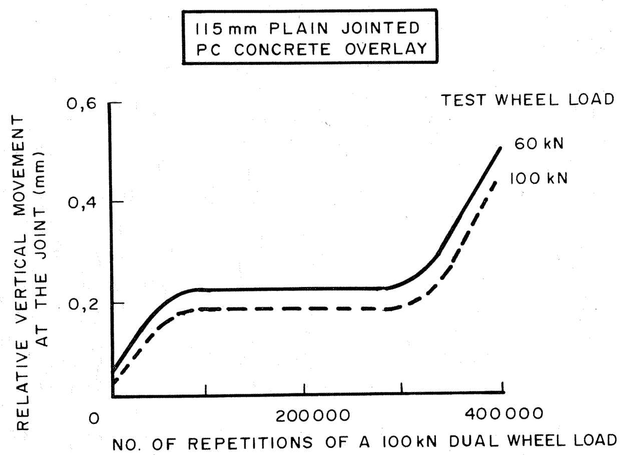


Figure 13. Vertical movement at different stages of trafficking.



position and speed of the wheels.

Instruments are available to measure surface deflection and the curvature of the pavement, the transverse profile of the road surface, elastic deflection and permanent deformation at different depths within the pavement (6), the density and moisture content of the pavement layers, relative vertical and horizontal movements at cracks (7) and to take other relevant measurements such as the temperature of the pavement layers, air temperature and rainfall. Use is made of microprocessors and computer software to record the HVS data for evaluation at a later stage.

The HVS has been used to investigate how the pavement breaks down under traffic and to identify the principal mechanisms of deterioration in performance. A previous HVS study in 1981 showed that the concrete pavement at that time had a residual life of between 0.3 and 1 million equivalent 80 kN axle loads (8). The pavement has since carried about 0.6 million equivalent axles without a significant decrease in riding quality, although some 30 % of the joints have failed (C value more than 0.70). The HVS is now being used to evaluate the different experimental sections and wheel loads of 40 kN, 60 kN, 100 kN and 200 kN are being used.

The Multi-depth deflectometer (MDD), used to measure relative deflection at different depths of the pavement structure, and the modified Benkelman beam provided valuable data to compare the analytical model with field trials. Deflections and relative movements for example were measured as the overlay was trafficked with the HVS.

Figure 13 shows an example of the change in relative movement at a joint under a 40 kN HVS wheel load as the pavement is being trafficked. It is clear that after relative vertical movement increased slightly, the condition then stabilised until cracking could be observed at about 300 000 load application.

To date four overlays have been tested namely the 40 mm open-graded asphaltic concrete with asphalt rubber as a binder, the 150 mm continuously graded crushed stone, and the 125 mm jointed concrete and the 100 mm continuously reinforced concrete overlay. Cracking was the predominant mode of failure for all the overlays while rutting was not significant.

Different loads were used in the testing program with the HVS and based on performance, the load equivalent factor was found to be in the order of 3.5 for the PC concrete overlays. Load equivalent is defined as (wheel load in terms of kilo-Newton):

$$\text{Load equivalent} = \left(\frac{\text{applied wheel load}}{40}\right)^{(3.5)} \quad (3)$$

This value of 3.5 compares favourably with the 3.62 previously found in Texas for continuously reinforced concrete pavements (9) and the 3.5 for a plain jointed concrete road in South Africa (8).

It is intended that all the experimental sections will be tested for final evaluation under accelerated testing.

Evaluation of performance

The measured number of load applications to failure as well as the response of the rehabilitated pavement can now be correlated with values of response variables determined with the analytical model. To synchronize measured and calculated response, the model can be calibrated by adjusting the load transfer capabilities of the joint and cracks together with changes in the stiffness moduli of the different pavement components. The model can then be used to determine the relative influence of design parameters such as changes in overlay thicknesses or material types.

As an example, Figure 14 shows a plot of the calculated maximum shear and maximum horizontal stresses in the concrete slab overlay under 40 kN wheel loading where a crack or a joint occurs in the overlay directly above a joint in the old pavement for both the jointed (JCP) and continuously reinforced (CRC) concrete overlays. It is clear from this figure that horizontal stresses tend to level out and reach a low value when the concrete overlays are about 125 mm thick for the CRCP and 150 mm for the JCP. Shear stresses are lower than horizontal tensile stresses for the JCP, but in the case of the CRC the shear stress is significantly higher than the tensile stress. The higher shear stress in the CRC can be explained by the fact that the longitudinal steel reinforcement acts as a load transfer device with a consequent increase in shear stress in the concrete

Figure 14. Stress in an overlay with joint/crack directly above old joint.

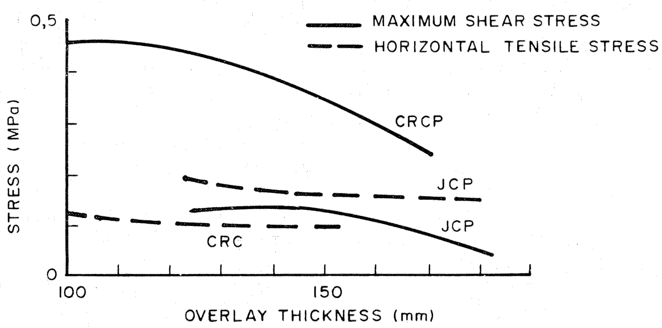
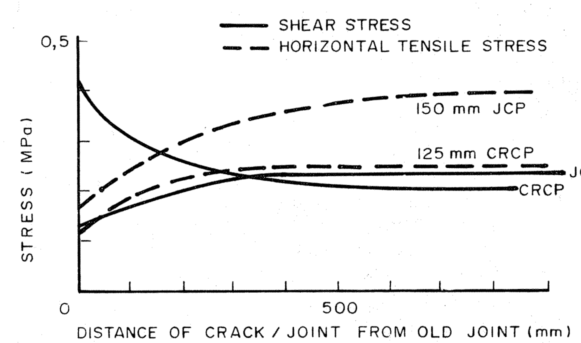


Figure 15. The influence on stress of the distance between a crack/joint in the overlay and the joint in the old pavement.



itself. This applies only where a crack in the overlay coincides with a joint in the overlaid pavement.

In order to illustrate the influence of the position of a crack or joint in the overlay in relation to the joint in the old concrete slab, a plot of stresses against joint/crack position is shown in Figure 15. The load in this case is placed directly above the joint in the old PC concrete pavement and maximum stresses are calculated in the overlay at this position. Both shear and tensile stresses in a 125 mm JCP and 100 mm CRC overlay are shown in this figure. It seems that the placing of a joint in the JCP or a crack in a CRCP within 300 mm of a joint in the overlaid pavement can have a significant influence on the stresses within the concrete itself. Except for the shear stress in the CRC overlay, the other stresses in both the CRC and the JCP tend to increase as the joint or crack in the overlay moves further away from the joint in the old pavement. A joint or crack in the overlay close to a joint in the overlaid pavement has a significant influence on the deflection of the pavement and the vertical stress on the subgrade. Values for both these variables are shown in Figure 16 for the JCP and CRC overlays. Both vertical subgrade stress and deflection are almost constant for the CRCP but increase in the case of the JCP overlay whenever a joint comes closer than 300 mm to a joint in the old concrete slab.

Temperature stresses have not been included in the above analysis. The reason for this is that the HVS test is being done under controlled conditions with little variation in temperature and the intention is to correlate field performance with calculated performance on final design. Temperature stresses will, however, be included in the final analytical model. Using the values of calculated stress as shown in Figures 14 and 15 and the concrete strengths that were measured when the overlays were built, the expected life until cracking of the overlays occurs can be correlated with that actually measured by the HVS. The ratio of strength/stress is normally used in this case:

Number of load applications to failure $N = K(S)^d$

where K and d are constants

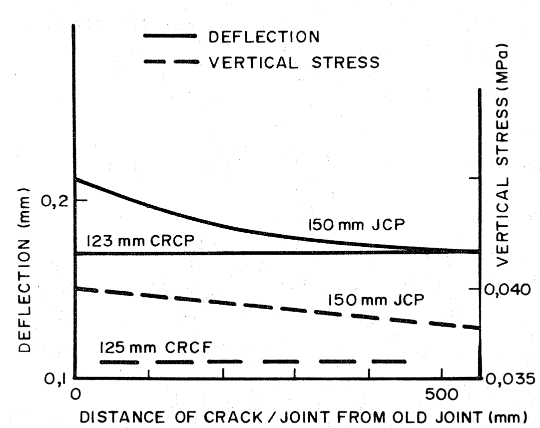
S is the strength/stress ratio

N is the actually measured number of load applications to failure.

Assuming that the load equivalency constant as determined previously can be applied in this case (i.e. that d has the value of 3.5) then K can be calculated if for example the laboratory determined flexural strength is taken as the strength parameter and the calculated tensile stress is taken as the limiting factor. When shear stress is considered in fatigue analysis, actual field performance has to be determined, which, it is hoped will be available for the CRCP in the near future, especially since the CRCP overlays have the higher shear stress. In the meantime, however, it can be assumed that shear strength is at least equal to tensile strength, and the above fatigue constants can therefore also be applied to shear fatigue.

The strains that result from a vertical subgrade stress as shown in Figure 16 may, however, cause some deformation of the subgrade, and according to experience on semi-rigid pavements in South Africa (5) they may result in a rutting depth of up to 4 mm after 2.5 million load applications. This figure has been confirmed with the HVS on the present site and the value can therefore be used to extrapolate to the other designs. If deformation of this magnitude occurs under a slab a loss of subgrade support or voids can be expected with subsequent pumping and an increase in slab stress with a higher risk of cracking.

Figure 16. Deflection and vertical stress on the subgrade as a function of distance of cracking/joint from the old joint.



The expected lives of some of the overlays are illustrated in Table 3. These values, 40 kN wheel load applications to failure, are based on the results of the analytical model as well as the HVS testing on some of these overlays. The minimum expected number of load applications has been calculated and the most critical condition in terms of cracking under tensile or shear stress or the deformation of the slab support system has been included in the table. It is expected however that the mean expected life can be as high as four times that shown in the table if the mean condition of the old pavement as well as the average characteristics are taken into account.

Table 3. Expected life of overlays.

Overlay	Expected Life (40 kN x 10^6)
150 mm JCP joints coinciding	3.0
150 mm JCP new joint 300 mm away	4.5
150 mm JCP new joint far away	4.7
125 mm CRCP crack at old joint	3.4
125 mm CRCP crack 300 mm away	6.5

According to the analytical model the mechanism of failure seems to be overstressing of the overlay support system, which causes deformation and a decrease in slab support, which in turn lead to higher stresses and thus cracking in the overlay. On site investigation on completion of the HVS test have indicated a small void between the flexible layer and the concrete overlay. It is not quite clear whether this void was caused by displacement of the flexible asphalt layer or by deformation of the whole slab support system. It nevertheless confirmed the postulation that a loss in subgrade support will eventually cause cracking in the concrete overlays. It must be emphasized that a pessimistic joint condition of between 0.55 and 0.70 has been used for the experimental sections and, in the actual rehabilitation of the pavement, minor repairs will be employed to improve these joints in a poor condition before overlays are being built.

Conclusions

Cracking of the plain jointed concrete pavement initiated as micro cracks and was due to the use of Portland cement high in alkali content and siliceous aggregate. Cracks initiated by this chemical reaction soon developed in structural cracking under traffic loading and, because of a loss in slab stiffness, stress on the sublayers increased which subsequently decreased their load-carrying capabilities.

In order to evaluate the condition of the pavement, it was visually surveyed using a C factor which relates to the load transfer capability of the joint. It was shown by experimental and theoretical comparison that these C values described the structural condition of the failed joint fairly accurately and could be related to deflection of the pavement and relative vertical movements at the joints. An analytical model was built to simulate measured pavement response such as vertical movement and deflection and this model was calibrated against actual field-measured response under accelerated loading using the HVS to simulate actual traffic loading.

Subsequently the analytical model could be used to calculate stresses and strains at different positions in the pavement and to relate these to laboratory-determined material performance curves. Monitoring of actual field performance was necessary to predict the lives of the different options in rehabilitation with greater confidence and experimental overlay sections were therefore constructed and trafficked with the HVS until failure. The performance data together with calculated response variables were then used to predict the performance of other types of overlays.

The following general conclusions can be drawn from this study:

1. PC concrete overlays are viable options for rehabilitation of the distressed pavement and will give at least 15 years of relatively maintenance free-life to the structure.
2. Failure of a rigid overlay under traffic loading will initiate as deformation in the sublayers and will consequently lead to a decrease in the support of the concrete overlay resulting in an increase in slab stress and thus cracking.
3. If a jointed concrete overlay is constructed, it will be better to provide joints in the new overlay as far away from the joints in the existing overlay as possible. This will decrease deflection as well as stress on the sublayers and will, it is hoped, extend the useful life of the slab support system. Since subgrade stress and deflection do not greatly depend on a discontinuity in a continuously reinforced concrete overlay, the position of the shrinkage crack is not very important.
4. If a shrinkage crack in a CRC overlay coincides with a joint in the old jointed pavement, high shear stresses can be expected in the CRC. These can be attributed to the load transfer capabilities of the longitudinal reinforcement, which in turn will cause high stresses in the concrete around the reinforcement close to the crack.
5. It is necessary to have a bond breaking layer between the old and the new layer of concrete to contribute to the prevention of reflection cracking. This bond breaker decreases high warping stresses as well as high stress concentrations in the overlay owing to discontinuities such as joints and cracks in the old rigid pavement.
6. Thicker overlays will perform better but a significant number of loads can be carried even with a 125 mm thick CRC overlay.
7. The use of a thin layer of asphaltic concrete with asphalt rubber as a binder seems to be a viable short-term solution which also fits in with the long-term management of the system since it can, if properly designed and constructed, be used as a bond breaker for future rigid overlays.
8. The analytical model can be used to evaluate other strategies and to extrapolate beyond the different options that were investigated on this site. It is, however, important to confirm the preliminary findings of the analytical study reported in this paper by trafficking the other experimental sections with the HVS and to monitor pavement performance with time in order eventually to include the effect of the environment on the performance of the different overlays.

Acknowledgement

This paper is published with the permission of the Directors of Bruinette Kruger Stoffberg Incorporated, the Directorate Land Transport and the National Institute of Transport and Road Research.

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