

Ben Schoeman CRCP: Findings from an Accelerated Pavement Test on the Shoulder of a 23 year-Old Pavement

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Abstract

Findings from an accelerated pavement test conducted with the Heavy Vehicle Simulator on the shoulder of a 23-year old Continuously Reinforced Concrete Pavement (CRCP) are presented in the paper. The Ben Schoeman Freeway connects the cities of Johannesburg and Pretoria in South Africa. The freeway is one of the most trafficked freeways in South Africa, with sections carrying up to 150 000 vehicles per day. It is the first and only freeway in South Africa where a CRC overlay was applied over the full width of the carriageway using a side form paver. The APT test comprised 2750 200 load repetitions predominantly at load levels of 80 and 100 kN, using a dual-wheel loading configuration. Environmental conditioning entailed the artificial watering of the test section for a two-day period, followed by six dry days. All other environmental conditions throughout the test were ambient. Pavement performance was characterised in terms of surface deflection, relative movement and permanent deformation at transverse cracks under repeated loading. Falling Weight Deflectometer tests were conducted before and after the test. The test section did not suffer structural failure through punchout. Diagnostic evaluation indicated the stripping and washing away of fines from the 30 mm open-graded asphalt overlay which formed the bond with the CRC.

Introduction

The Ben Schoeman freeway is a 21-km section of the national route N1, located in Gauteng, South Africa, between the Buccleuch Interchange linking the N3, N1 and M1 in the south, and the Brakfontein Interchange linking the N14 and N1 in the north.

The freeway is the second-highest trafficked road in South Africa, with the current average annual daily traffic varying between 116 000 and 159 000 vehicles along the length thereof. The heavy vehicle component of the traffic stream varies between 5000 and 6 500 vehicles per day in both directions, with an approximate directional split of 50:50.

The freeway was constructed from 1967 to 1969, originally as a four-lane, dual carriageway road with a flexible pavement consisting of a cement-stabilized subbase, crushed stone base and thin asphalt surfacing. During 1973/4 the riding quality of the pavement was corrected by a continuously graded HMA of varying thickness, followed by a 30 mm thick open-graded HMA overlay.

The freeway was upgraded during 1987/ 8 to a six-lane dual carriageway road by widening both carriageways in the median, using a 150 mm thick continuously reinforced concrete pavement (CRCP). The existing pavements of both carriageways were strengthened by a 185 mm thick continuously reinforced concrete (CRC) overlay. The CRCP was originally designed for a period of 30 years with periodic maintenance at year 15 consisting of a bituminous overlay, including a Stress Absorbing Membrane Interlayer.

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Periodic maintenance was carried out during 2005, with localised patching; the sealing of longitudinal and transverse joints and cracks wider than 2.0 mm; and stitching longitudinal joints showing transverse movement.

The South African National Roads Agency Limited (SANRAL) implemented the Gauteng Freeway Improvement Project (GFIP) as a far-reaching program for the province's major freeway networks. One contractual package entailed the rehabilitation and capacity upgrade of the Ben Schoeman Freeway, starting in May 2008.

The rehabilitation of the existing pavement under the GFIP was implemented to ensure a maintenance-free decade. The project consisted of:

- Localized patching by means of full-depth replacement of CRC;
- Sealing of all longitudinal and transverse joints and cracks wider than 2.0 mm by SBS polymer-modified sealant;
- 13.2 mm single seal using bitumen-rubber filler as a stress-absorbing membrane interlayer (SAMI) and to seal all cracks finer than 2.0 mm; and
- 35 mm nominal, semi-open graded, hot-mix asphalt overlay with bitumen-rubber binder (BRASO) to improve the riding quality and skid resistance.

The project also includes the capacity upgrading of the freeway consisting of either remarking and/ or widening the CRCP on the outside to obtain a minimum eight-lane dual carriageway configuration.

Traffic loading up to 2007 indicates that the highway in the vicinity of the test section have carried 26.0 million equivalent 80 kN standard axles (MESAS). The future design traffic loading study (BKS, 2009) envisage a traffic loading of between 67.0 and 87.0 MESAS at the end of the 30-year design life.

The need to confirm the estimated remaining life of the existing pavement was considered paramount to the detailed design process, as it would serve to inspire confidence in the proposed pavement rehabilitation and strengthening measures. Accelerated Pavement Testing (APT) was considered as the most appropriate technique to simulate the material behavior and to evaluate pavement performance under wheel loading in a relatively short time period.

With this as a point of reference, as well as the framework of the current and ongoing refinement and upgrading of the concrete pavement performance models as used in the *cncPave* design software, the objectives of the research project were to:

- Characterize CRCP performance in terms of standardized measurements for surface deflection and relative movement of transverse cracks under repeated loading and Falling Weight Deflectometer; and
- Contribute to the current body of knowledge through the development of a performance model for the section tested.

Given the background to the project; this paper serves to discuss the APT test and pavement responses as stated in the first objective of the research study. The latter objective is briefly touched upon; the developed methodology is presented in depth in a publication by De Vos (2012).

Accelerated Pavement Testing

Test Site. The site for the APT was chosen after considering the scheduled construction works and the lane configuration along the freeway. An area for testing was selected at km 10.260 on the southbound carriageway of Road N1 Section 21.

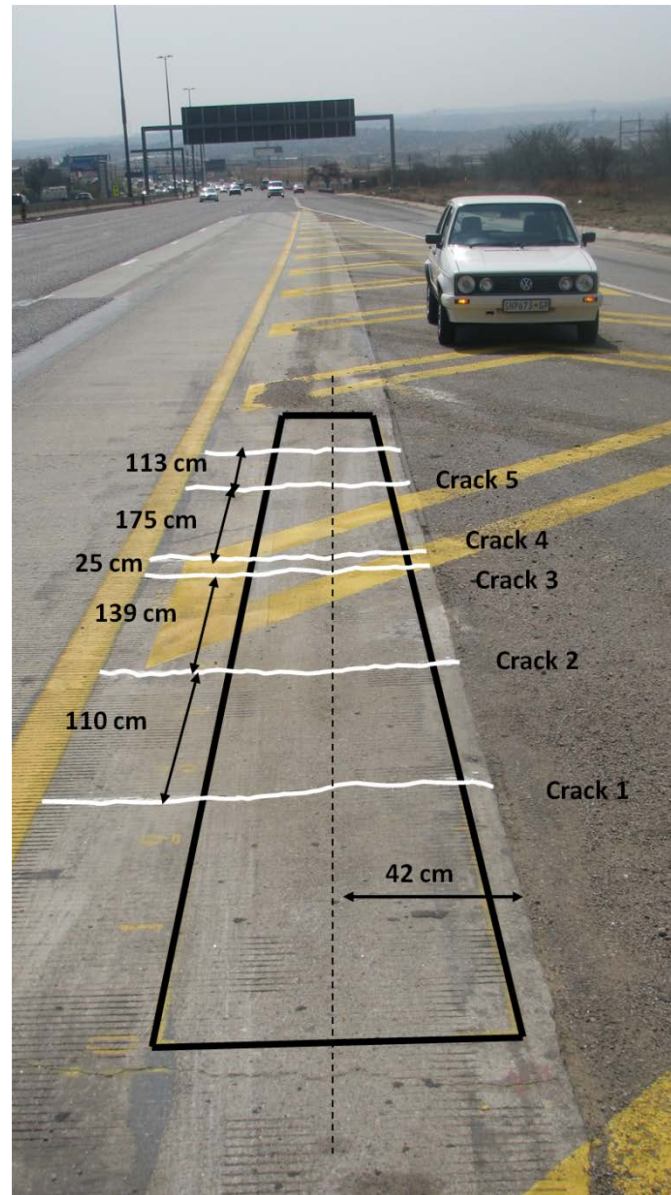


Figure 1: View of the HVS test section

The test section was located on the free untied edge of an unused 1.5 m wide CRC shoulder, within the exit gore area of the Brakfontein Road off-ramp of the Rooihuiskraal I/C. The edge is abutted upon the flexible pavement of the adjacent off-ramp; the 30-mm space between the two pavement structures is filled with a rubber sealant. Apart from environmental loading, the section was deemed as untrafficked based on the location and surrounding road geometry.

The test section was representative of poor or vastly deteriorating sections on the freeway. Based on the backcalculated modulus of subgrade reaction, the general condition of the test section was at that of the 20th percentile of the Ben Schoeman CRCP before the start of accelerated testing. In the eight-meter section there are six transverse cracks of which five were instrumented. The average spacing of the transverse cracks was 1090 mm. Two transverse cracks at the middle of the section were at a distance of 245 mm from each other. The proximity of these two cracks made the test section susceptible to punchout through the formation of a longitudinal crack between the closely spaced transverse cracks.

The section was selected for the purpose of traffic loading close to the pavement edge. Edge-loading induces significantly higher pavement stresses, which leads to accelerated deterioration. The outer side of the loading tire was on average 100 mm from the edge of the pavement.

The structure of the test section, as summarized in Table 1, is representative of the freeway sections under consideration.

Table 1: Test Section Pavement Structure

Layer thickness (mm)	Material	Year constructed
185	Continuously reinforced concrete	1987
30	Open-graded asphalt overlay	1973
30	Continuously-graded asphalt	1969
200	G1 - Crushed stone base	1969
150	C2 - Stabilized gravel subbase	1969
150	G7 - Selected subgrade	1969
150	G9 - Selected subgrade	1969

Accelerated loading. Using the Heavy Vehicle Simulator (HVS), testing was conducted by the Council for Scientific and Industrial Research (CSIR), and was done from 17 December 2009 to 12 May 2010. A total of 2750 200 half-axle loads were applied in canalized, bi-directional trafficking mode.

The applied loads comprised load levels of 40, 60, 80 and 100 kN respectively. Loading was applied with a dual-wheel configuration of two 12R22.5 tires at pressures of 800 kPa.

Load levels of 40 and 60 kN were used for the first 80 000 applications, followed by loading at 80 kN. After 1730 500 applications, pavement evaluations revealed that no noticeable degree of deterioration had started to take place and a decision was made to increase the load level from 80 to 100 kN.

To simulate rainfall the section was artificially and continuously watered for a two-day period, followed by six dry days. Wetting of the section entailed the continuous dripping of five liters per hour on the test section. The watering/ dry cycles started after the initial 200 000 loads; and a total of 16 cycles were completed during testing. All other environmental conditions were ambient throughout the test.

Instrumentation. Maximum surface deflection and permanent surface deformation were measured by means of Joint Deflection Measuring Devices (JDMD's). JDMDs are linear variable displacement transducers (LVDTs), mounted vertically on the concrete slab. The relative movement across a transverse crack, with the entire contact area of the loading wheel on one side of the crack, was calculated from the JDMD data. Horizontally-mounted JDMD's at Cracks 2, 3 and 5 measured slab contraction and expansion due to daily temperature fluctuations and loading. The normal resolution obtained from the LVDT's used is 4 micron. The maximum linearity error for the LVDT sensor is $\pm 0.25\%$ of full range output.

Thermocouples were installed to measure environmental and pavement temperature, placed at the surface (0 mm) and at the bottom (175 mm) of the CRC. A

weather station at the test site measured air temperature, barometric pressure and relative humidity. Instrumentation was tethered to a central computer that logged the pavement responses and environmental measurements at 30-minute intervals. A total of 6953 sets of pavement responses and environmental conditions were measured.



Figure 2: Horizontal and vertical JDMDs and Cracks 3 and 4 after 1.6 million load applications

Twice-daily, at 05:00 and 13:00, pavement responses were measured under standard loads. In the case of 80 kN trafficking, the standard loads were 40 and 60 kN; in the case of 100 kN trafficking the loads were 40 and 80 kN.

Pavement Responses

Surface deflection. Maximum pavement surface deflections, measured by the JDMDs, are illustrated in Figure 3 for Cracks 3 and 5 respectively. The visible variation in the deflections is explained by the twice-daily measurements at standard test loads of 40 kN. This variation due to the loading schedule is superimposed on the daily variation due to changing environmental effects.

The vertical line indicates the change of the test load from 80 kN to 100 kN. The rectangles in the background indicate watering cycles.

The daily 40 kN deflections, taken at 05:00 and 13:00, together with the surface temperature and temperature differential (difference between the surface and bottom temperature in the CRC) are illustrated in Figure 4. These figures clearly illustrate the inverse relationship between the deflection performances and temperature regime.

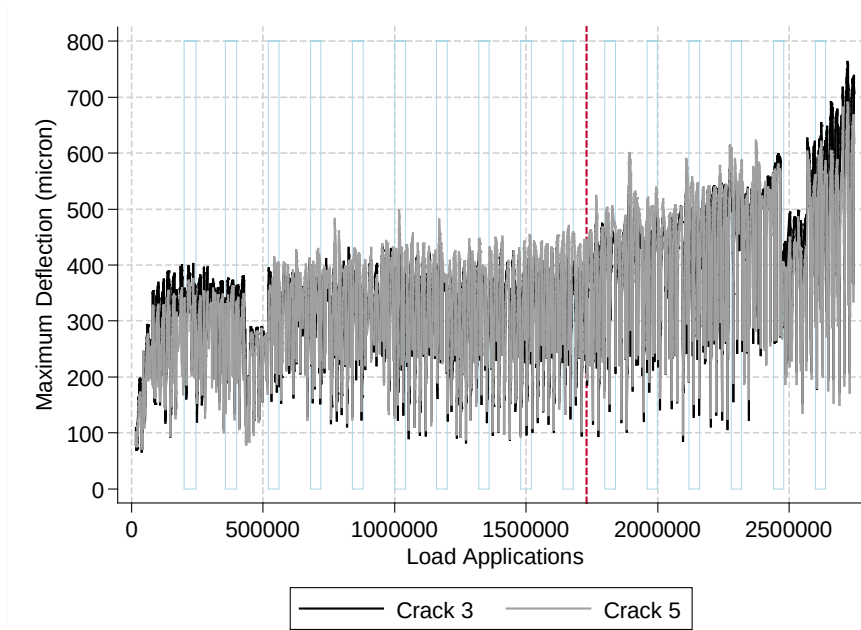
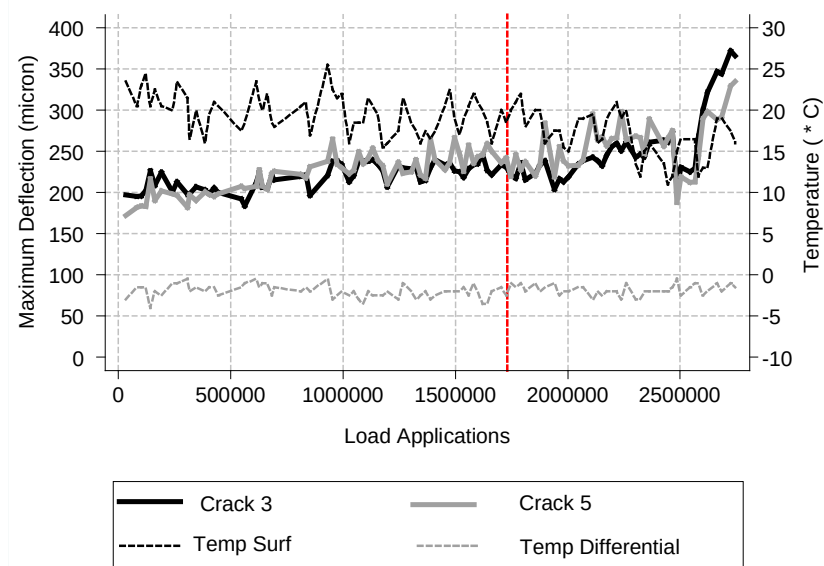
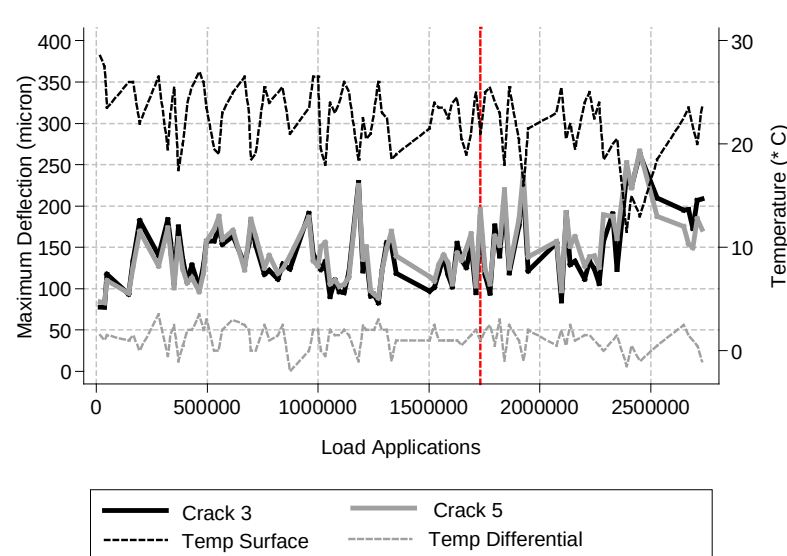


Figure 3: Maximum deflections at Cracks 3 and 5 as measured at 30-minute intervals



(a)

Figure 4: Daily deflections taken at 05:00 (a) and 13:00 (b) under 40 kN



(b)

Figure 4: Daily deflections taken at 05:00 (a) and 13:00 (b) under 40 kN

Horizontal movement. A typical horizontal crack movement measurement is illustrated in Figure 5. A vertical JDMD measurement is also presented for each crack to illustrate the relative location of the loading wheel to the transverse crack. The horizontal JDMD registers a positive deflection when the two crack surfaces move away from another. The crack opening occurs when the loading wheel is at some distance away from the crack.

As the wheel approaches the transverse crack, the measurement declines and becomes negative. The horizontal deformation is at a minimum when the wheel is exactly above the crack. Whether the crack is completely closed or still open when the loading wheel is on top of it is not known, as the measurement frequency was too low to follow the procedure proposed by Kholer (2005).

The correlation between the closing and opening of the three cracks monitored horizontally vary between 0.265 and 0.94, based on a sample of 6953 observations.

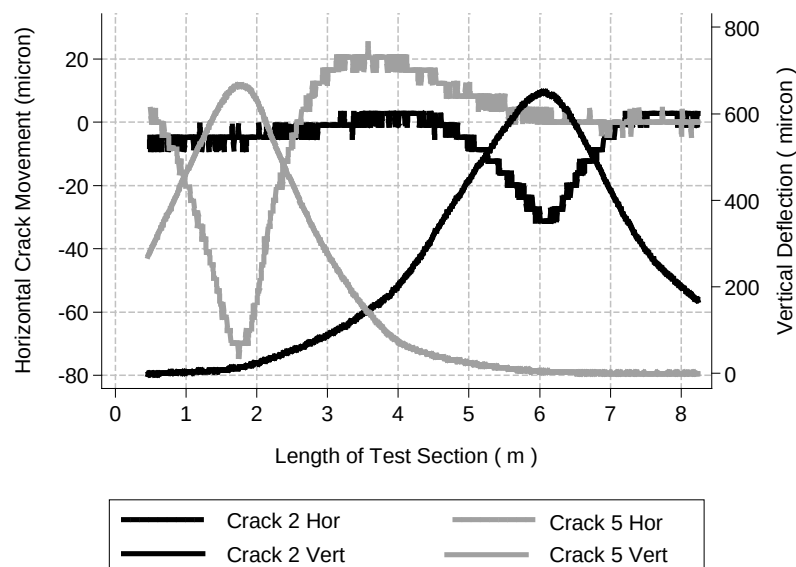


Figure 5: Horizontal opening and closing of transverse Cracks 3 and 5

Relative movement. The relative movement across transverse cracks under wheel loading was determined from the deflection data. Figure 6 illustrates the relative movement of Cracks 3 and 5; the variation in relative movement is also due to the twice-daily standard measures, superimposed on daily environmental fluctuations.

The calculation was conducted with the complete footprint of the loading wheels on one side of the transverse crack, using the size of the tire footprints under all the different loading conditions – 40, 60, 80 and 100 kN. This was done for the approach and leave slabs, as the maximum deflection and hence relative movement were not consistently on the same sides of the transverse cracks for the loading regime used.

Correlation coefficients of total horizontal movement and relative movement vary between 0.57 and 0.82 for the three cracks instrumented.

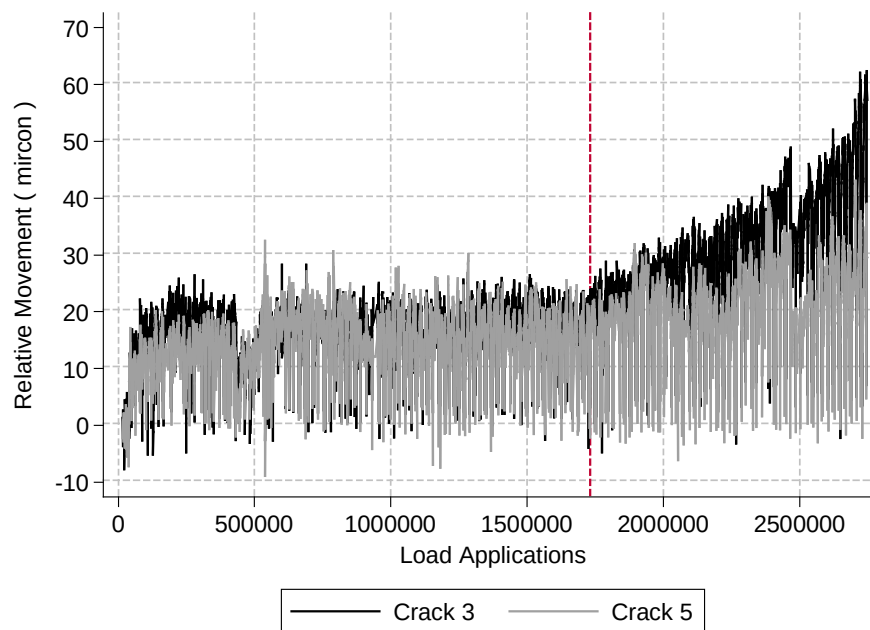


Figure 6: Relative movement for Cracks 3 and 5 at 30-minute intervals

Permanent deformation. The normalized permanent deformation measured at the side of the crack that exhibited the highest deterioration is illustrated in Figure 7. The short-term cyclical nature of the permanent deformation plots are due to the warping and curling caused by the daily environmental fluctuations.

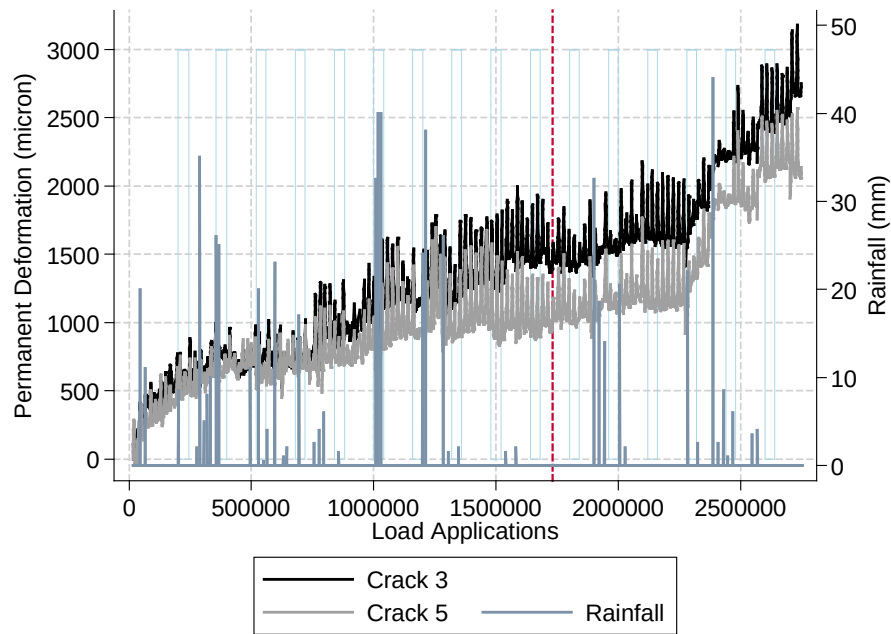
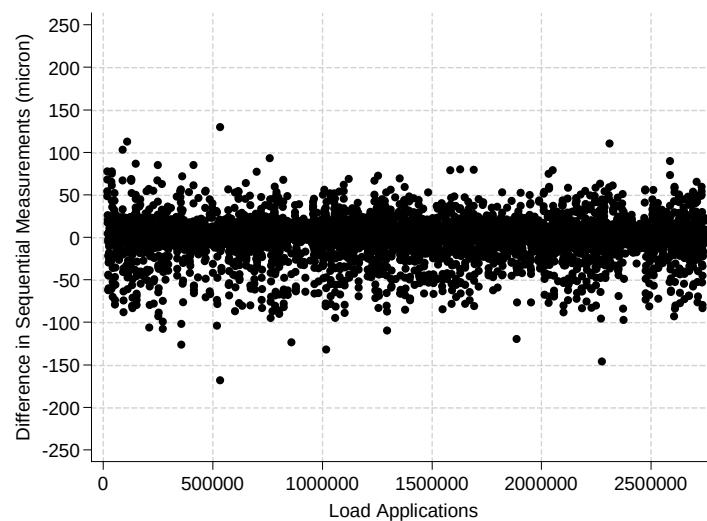


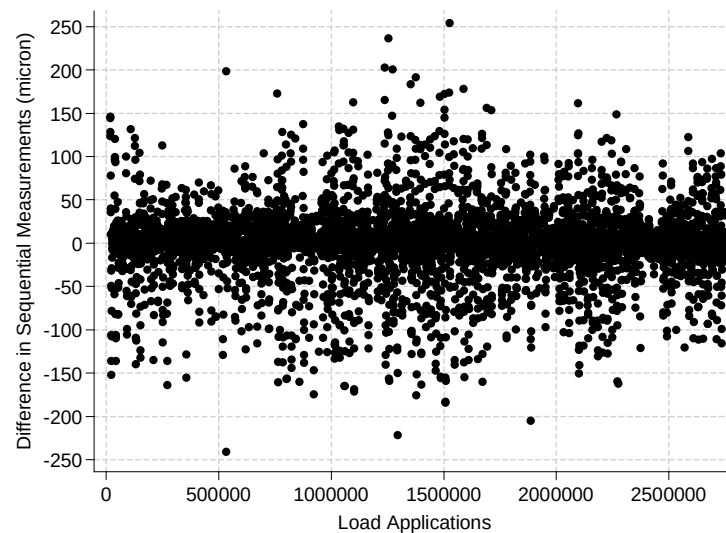
Figure 7: Normalized permanent deformation for Cracks 3 and 5 at 30-minute intervals

Sequential changes in the permanent deformation measurements are illustrated in Figure 8. The figures present the changes due to warping and curling experienced by the concrete superimposed upon the measured actual permanent deformation. Crack 1 exhibited a very low variability compared to that of Crack 4. The variation of the scatter for the period when the permanent deformation experienced a gradual shift, 2 300 000 to 2 400 000, does not correspond to the variation of the scatter for the rest of the test.



(a)

Figure 8: Scatter of sequential permanent deformation changes:
(a) Crack 1, (b) Crack 4



(b)

Figure 8: Scatter of sequential permanent deformation changes:
(a) Crack 1, (b) Crack 4

Visual observations. During the course of the test the pavement showed very few visual signs of distress. Evaluation of the pavement condition after 1.6 million 80 kN load applications indicated no noticeable degree of deterioration since commencement of the test. It was consequently decided to intervene in order to attempt to introduce some degree of deterioration.

The load level was increased to 100 kN. No noticeable pumping from any of the cracks were detected despite the nine watering cycles and 416 mm rain that fell up to that point in time. Transverse cracks after 1662 893 load applications are depicted in Figure 2.

After 2.6 million load repetitions and 16 wet cycles, there was still no sign of serious deterioration. Apart from the added surface water, there was additional rainfall of 216.5 mm. The only sign of deterioration was a small degree of spalling on the northern side of Crack 4; not very different from the spalling visible in Figure 2 after 1.6 million load repetitions. After this evaluation on 5 May 2010, the test was terminated on 12 May 2010 and the site was evacuated in compliance with the 2010 FIFA Soccer World Cup trafficking and road policy.

Diagnostic investigation

Diagnostic evaluation by means of coring took place after testing. Five 150 mm-cores were extracted at the positions of the transverse cracks; in such a way that the crack bisects the core. Prior to taking the 150 mm core, a 50 mm core was drilled to a depth of 200 mm. The hole was subsequently filled with a high-viscosity epoxy tinted with red iron oxide. The objective of the methodology was for the epoxy to penetrate and fill a possible void below the CRC.

The procedure had mixed success due to the viscosity and flow characteristics of the epoxy and the disintegration of the top open-graded asphalt layer. The top structure of the pavement consists of 185 mm CRC; 30 mm open-graded asphalt overlay; 30 mm continuous-graded asphalt surfacing; and a 200 mm crushed stone base.

Evaluations of the extracted cores indicated that the asphalt overlay started to strip and lose fine material. The extracted cores presented different stages of HMA stripping and loss of fines. The loss of fine material did not, however, materialize in pumping observed at the surface at this stage of deterioration.

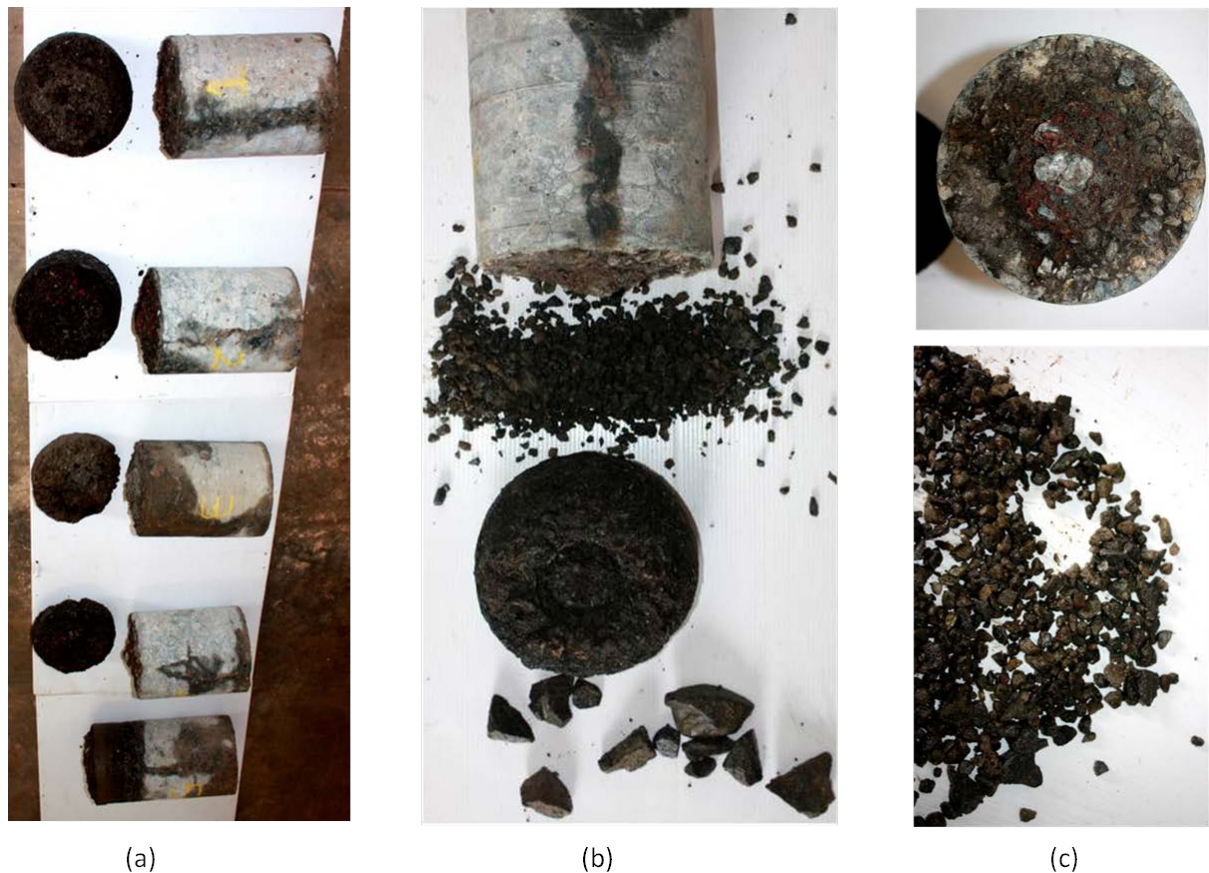


Figure 9: (a) Five extracted cores, displaying various degrees of distress, (b) decomposition of extracted pavement layers and materials - Core 4, (c) CRC-HMA interface and loose and stripped aggregate - Core 4

Crack widths measured by means of feeler gauge are summarized in Table 2. Measured crack widths decrease with core depth.

Table 2: Measured Transverse Crack Widths in mm

Position of measurement	Transverse crack number				
	1	2	3	4	5
Surface of core	0.840	1.080	1.430	0.730	0.630
Side of core - top	0.535	0.290	0.980	0.365	0.440
Side of core - middle	0.220	0.215	0.515	0.265	0.115
Side of core - bottom	0.080	0.215	0.510	0.155	0.105

Supplementary Testing: Falling Weight Deflectometer

FWD testing prior to the arrival of the HVS on site took place on 14 December 2009, and HVS testing started on 17 December 2009. The FWD tests after completion of the accelerated loading were conducted on 23 October 2010. It should be noted that the

traffic loading stopped on 12 May 2010, five months prior to and in a different climatic season to the FWD testing.

Effects of edge loading. The effects of edge loading were investigated by means of FWD after accelerated loading. FWD measurements were taken at various distances from the pavement edge for a load spectra ranging from 40 to 120 kN. These measurements were taken on top of Crack 1, with the loadplate placed symmetrically on the transverse crack. This test could not have been conducted on an untrafficked section in the vicinity as all the sections had already been sealed.

The objective of this approach is to obtain an indication of the effects of edge loading on an unbounded edge of a CRCP. Although the effects of edge loading are quantified in terms of deflection, it should serve as an indication of the stress level in the pavement structure. A three-dimensional deflection basin for the 40 kN load case is illustrated in Figure 10.

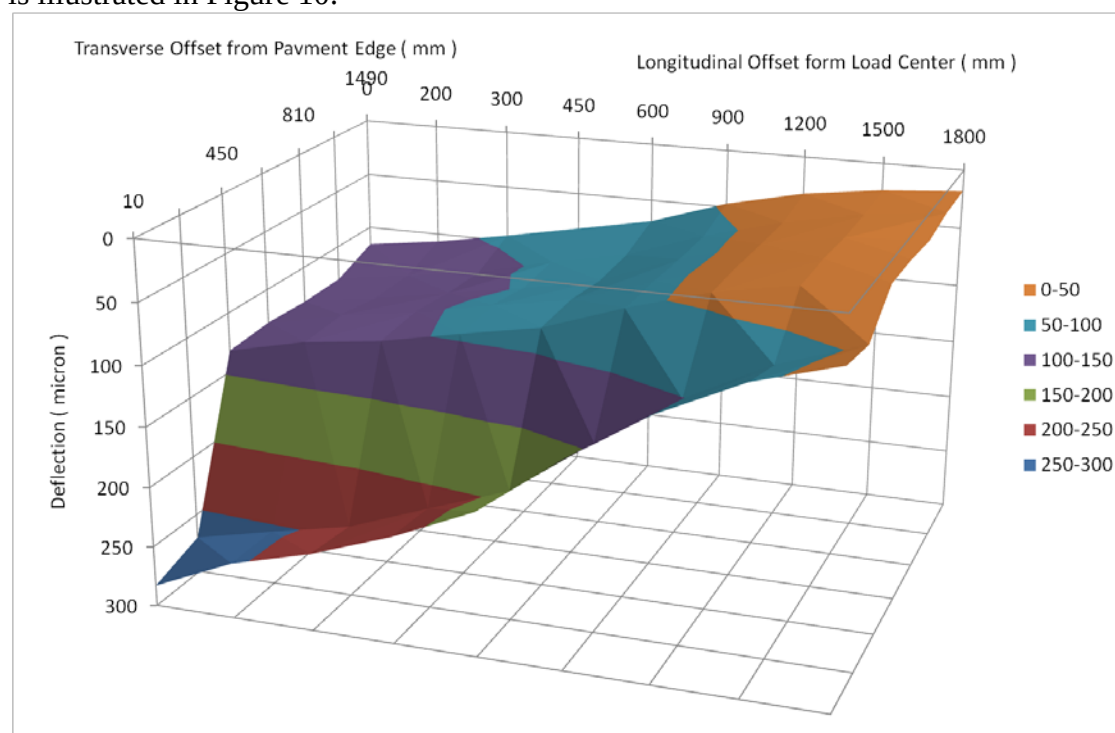


Figure 10: Thee-dimensional plot of measured surface deflections at offsets from CRCP edge

Backcalculation of pavement parameters. FWD data was used for backcalculation of pavement parameters and compared to the distribution found for the Ben Schoeman Freeway from a FWD survey conducted in 2006. Based on the relative modulus of subgrade reaction, the test section before and after accelerated loading is on the 20th and 7th percentile of the distribution from the survey.

Stable long-term relation between relative movement and deflection

The analysis methodology proposed by De Vos (2011) was applied to the pavement response data. A quick overview and the results are subsequently presented.

There is a demand for quantifying the long-term equilibrium relationship between pavement performance and failure mechanisms. A statistical property of non-stationary time series can be applied to pavement response data to quantify the long-run equilibrium relationship between mechanisms of pavement failure and

deterioration. The statistical property is that of cointegration, which allows for testing for the existence of equilibrium relationships among trending variables within fully dynamic specification frameworks.

In the case of a thin (150 – 185 mm), continuously reinforced concrete pavement, punchout, the predominant failure mechanism, is caused due to the loss of load transfer capability at transverse cracks and the deterioration of the support structure (Selezneva et al. 2004; Strauss and Lourens 1998; Steyn et al. 2005 and Brink 2003). The analysis methodology allows one to quantify the interdependence of these mechanisms.

It is postulated that a statistically significant, stable and long-term linear relationship exists between deflection, relative movement and permanent deformation, as illustrated in Figure 11. These three series serve as the proxies for pavement response, the load transfer capability at transverse cracks and the deterioration of the support structure below the CRC respectively.

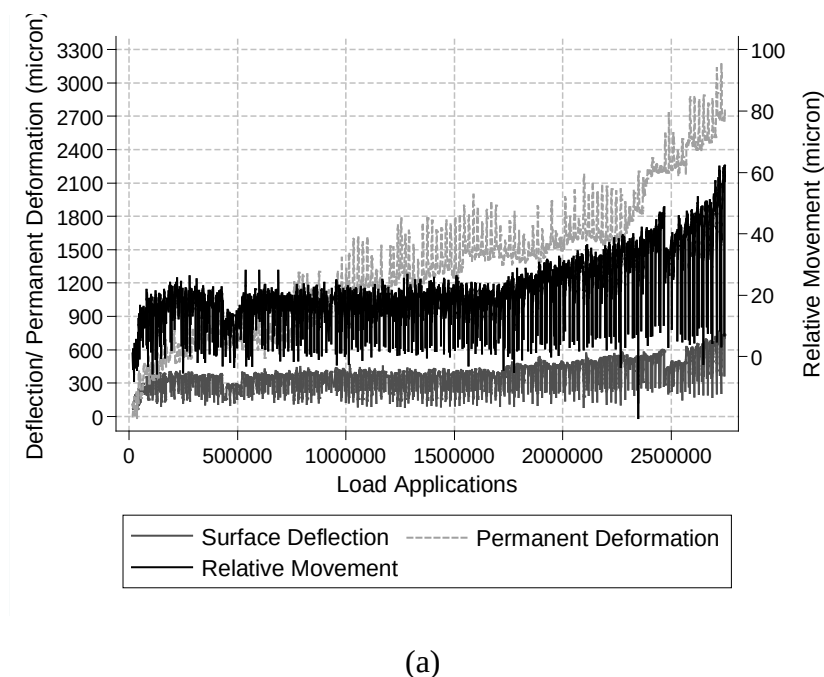
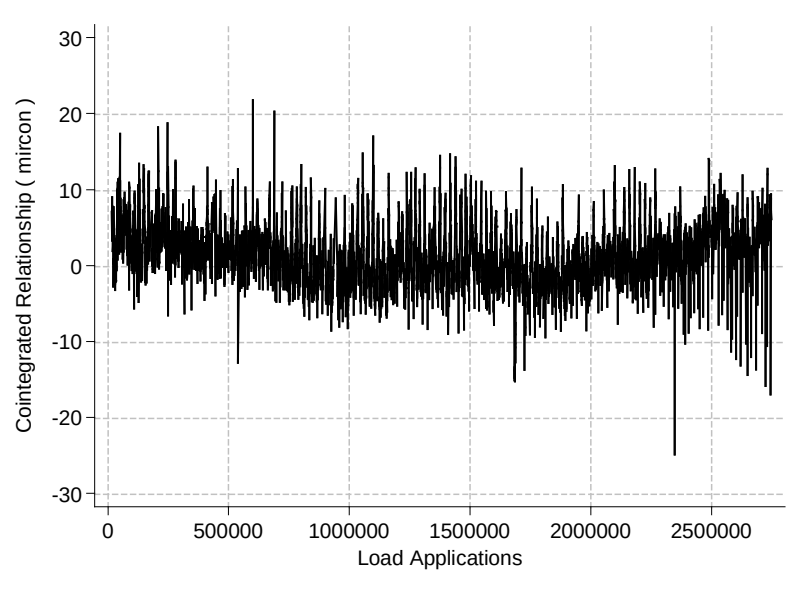


Figure 11: (a) Permanent deformation, dynamic deflection and relative movement of Crack 3; (b) the cointegrated relationship fluctuating around zero



(b)

Figure 11: (a) Permanent deformation, dynamic deflection and relative movement of Crack 3; (b) the cointegrated relationship fluctuating around zero

The full specification, relating deflection to both permanent deformation and relative movement, breaks down due to permanent deformation being a poor proxy for the deterioration of the substructure (De Vos 2011). As a result a simplified model, Equation 1, was estimated that quantifies the long-term relation between deflection (jd) and relative movement (rm), with c the constant and ε the stochastic error term. The implied long-run equilibrium relationship for each crack is summarized in Table 3, where the accent on a variable, e.g. $\hat{c}$, indicates that it is an estimated parameter.

$$jd = c + \beta rm + \varepsilon \quad (9)$$

The estimated coefficient should be viewed as elasticities in an economic sense. It could be presented as partial derivatives of Equation 1. The average long-term relationship between a change in the relative movement (rm) and a change in maximum surface deflection (jd) for the section tested is 17.5. The relationship quantifies the impact of a loss of load transfer capability on surface deflection and ultimately pavement performance.

Table 3: Coefficients of the Long-Run Relationships

Crack nr	$\hat{\beta}$	$\hat{c}$
1	16.93	0
2	26.114	189.29
3	9.909	159.218
4	20.025	135.595
5	14.678	200.378
Average	17.533	136.897
Standard Deviation	6.051	80.647

Conclusions

This paper summarizes the results obtained from the APT test on the 23-year old CRC Ben Schoeman freeway. The test was conducted to evaluate pavement performance in terms of standardized measurements under repeated loading. Although the test did not lead to pavement failure through punchout; a marked deterioration in pavement performance measures were observed.

Based on the findings presented, the following conclusions are drawn:

1. The increase in the load level from 80 to 100 kN accelerated pavement deterioration as measured by surface deflection and relative movement across cracks. Surface deflection increased at an increasing rate. Relative movement was at a plateau for 1.2 million repetitions before the intervention; after which there was a marked and continued increase. The increase in load level did not have such a marked impact on the permanent deformation, as it continued at the trend prior to the intervention.
2. The deterioration of the bond between the CRC and the substructure entailed hairline cracking followed by stripping of the open graded asphalt layer. The fines and stripped material moved laterally within the pavement as no pumping of fines to the surface was observed.
3. The backcalculated modulus of subgrade reaction, before and after APT, indicated deterioration of the pavement substructure during accelerated loading.
4. The elasticity of CRC surface deflection with regard to the relative movement of a transverse crack is on average 17.5. It states that a 1.0 % increase in relative movement would translate into a 17.5 % increase in surface deflection and vice versa. This quantifies the average long-term relation between a change in relative movement and a change in maximum surface deflection.
5. The total number of standard 80 kN axle loads that were simulated amount to 78.4 million if one applies the 4.2 power damage law to the loading regime. This in itself is a crude estimation and does not take into account the effects of edge loading and age of the pavement. It is in line however with the upper limit of the envisaged design life of 78.0 MESAS.

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