Performance of a Thin CRCP Inlay Designed for a Five-Year Life: a Case Study

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Abstract

The steep uphill section of an interstate highway had experienced considerable damage in the form of serious rutting due to slow-moving heavy vehicles. The asphalt pavement had had to be rehabilitated every 4 years and a decision was made to mill and replace the top 180mm of the slow lane with a CRCP inlay. The inlay design was based on an anticipated 6 million equivalent 80 kN axles over five years and it was envisaged that a concrete overlay would be placed over the full width of the road at the end of this period. However, at the end of six years only approximately 0.25% of the area of the inlay had shown serious distress. It was decided to determine the remaining structural life of the pavement with a view to repairing failed areas permanently, rather than temporarily, if the remaining structural life was found to be significant.

This paper describes the testing of behaviour and performance on a relatively highly cracked section of the inlay using the Heavy Vehicle Simulator (HVS). Although the test section had narrowly spaced transverse cracking prior to testing, no punch-outs could be created using the HVS. A detailed survey of the 26 lane km of inlay showed that failure did not necessarily occur at closely spaced cracks, but was more associated with edge loading, loss of slab support and poor quality concrete. It also became very clear that a high variation in concrete characteristics (and in particular concrete strength) resulted in a high risk of structural failure.

The results were translated into transfer functions that have been included in cncPave, a mechanistically-based design method for concrete pavements developed for South African conditions.

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Background

Concrete pavements in South Africa, especially Continuously Reinforced Concrete Pavements (CRCP), have in the past been designed primarily using design methods developed and used in the USA. Several of these sections have been and are still being monitored for their performance and the information is being used in the upgrading of design methods developed since 1990 for South African conditions. The design method presently being used, cncPave, is a mechanistically-based design method which also uses stochastic principles to predict the extent of structural failures with time. However, it was found that the performance models used in cncPave did not always render accurate predictions of the extent of failure and therefore needed refinement.

The construction of jointed pavement test sections at Hilton in KwaZulu-Natal and the testing of their performance under Heavy Vehicle Simulator (HVS) traffic, presented the opportunity to determine the remaining structural life of a thin CRCP inlay on the N3, close to the test sections. The HVS is used for accelerated pavement testing and is capable of applying up to 20 000 wheel loads per day, ranging from 40kN to 100kN. The CRCP inlay on the N3, 180mm thick, containing 0.61% longitudinal steel, on a 50mm layer of asphaltic concrete over a 150mm crushed stone layer, had been constructed some six years previously using labour intensive construction methods. It had already carried its design traffic loading. The use of the HVS on a section of the inlay on N3 where structural failure seemed to be imminent was an ideal opportunity not only to refine the performance models, but also to establish the remaining life (and thus the performance) of such inlays. In this way, a better understanding of the link between HVS performance predictions and actual performance under real traffic could be created. Since the HVS testing was limited to one suitable site, some additional evaluations were carried out on the rest of the inlay on the N3 close to Durban to establish the actual performance of this pavement. In this paper, only limited data and analyses from a very extensive testing program are presented.

Condition of the thin CRC inlay

The CRC inlay was designed to carry 6 million equivalent 80kN (E80) axle loads and several punch-outs had occurred after approximately 7 million E80 axle loads. Figures 1 to 3 illustrate the type of distress encountered on the pavement which was found to lead to the development of punch-outs as typically experienced on the CRC inlay. Figure 1 shows the most likely location where a punch-out is to be expected, namely at a construction joint where closely spaced and erratic cracking occurs at a permeable edge joint between the concrete inlay and the adjoining flexible pavement. The cracks are normally associated with relatively poor concrete, possibly caused by segregation, excessive fines, excessive water in the concrete or adverse environmental conditions during construction. As soon as the cracks start showing spalling, relative vertical movements occur under a wheel load moving across the cracks, initiating a loss of load transfer and a loose block develops as shown in Figure 2. This is defined as a punch-out since the block will become loose and create a safety hazard. Figure 3 shows the development of a longitudinal crack down the middle of the travelling lane as a result of a combination of a loss in slab support at the edge of the pavement and wheel loading close to the edge of the inlay.

Figure 1. A cluster of cracks at a transverse construction joint and an open edge joint

Figure 2. A cluster of cracks, an open edge joint and edge loading lead to a punch-out

Figure 3. Longitudinal cracks at clustered transverse cracks can lead to punch-outs

The total length of CRC inlay (12 km) was surveyed for signs of structural distress. Table 1 summarizes the extent of distress in the slow lane of both the north and south bound carriageways surveyed in 2004, six years after construction. The table includes the area of potential punch-outs i.e. where a punchout is to be expected.

Table 1. Distress in the CRC inlay of the N3, Pietermaritzburg by-pass

Lane	Cracks[1]		Potential punch-out	Punch-out[3]
	Cluster[2]	Longitudinal		
South bound	1.9%	18.3%	0.4%	0.5%
North bound	2.0%	13.8%	0.4%	0.3%

Notes: 1. The percentage of cracks is indicated in terms of the length of pavement affected.
2. Cluster refers to a cluster of transverse cracks
3. The extent of punch-outs was established by assuming that an area of $4m^2$ of pavement has been affected per punch-out and will have to be repaired.

HVS testing of the thin CRC inlay

The CRC inlay on the N3 near Pietermaritzburg was constructed in 1998 and has to date (December 2004) carried about 7 million E'80s over the six years. It was originally designed for slightly less traffic. Only a few punch-outs had occurred (see Table 1), mainly at transverse construction joints and close to the inner longitudinal edge where the road is in a horizontal curve.

In order to obtain an indication of remaining life, a section was selected for testing with the HVS in a location where the existing crack pattern suggested the potential of a punch-out developing. As shown in Figure 4, cracks occur in a cluster with some secondary cracking already occurring – suggesting the possibility of imminent failure.

The pavement was subsequently tested using a 60 kN wheel load applied bi-directionally, first under normal moisture conditions, then followed by the introduction of water to wet the unbound support layers. Later a wheel load of 80 kN together with the introduction of water was used.

Figure 4. Test site for HVS testing

The HVS testing program included measurement of in-depth deflections, permanent deformation, relative movements at cracks and temperature variations as well as Falling Weight Deflectometer (FWD) measurements before and after the HVS testing. Table 2 summarizes the data that was captured over the duration of the test programme. Details of the data as well as other information obtained from this testing program are contained in the first-level analysis reports (du Plessis 2004, Brink and du Plessis 2004).

The data in Table 2 shows the range of some of the readings that were obtained as a result of the daily variation in both temperature and humidity. Note also that, at the start of the test, the deflections were essentially the same for the top of the concrete and the top of the base below the concrete. As the number of loads increased, the deflections at the top of the base decreased, but the concrete surface deflections increased, which is an indication of a gap developing between the concrete slab and the top of the base. The relative vertical movement at the crack also increased with an increase in the number of load applications. However, little additional visual distress could be detected after 6.5 million equivalent 40kN wheel loads had been applied by the HVS and the only visual sign of distress was very slight spalling that had occurred along the crack.

Table 2 Summary of data CRC inlay under HVS loading

Load applications		Deflections (mm)		Crack movements (mm)
Magnitude	Number	Surface	Top of Base	
60 kN	start	0.08 to 0.16	0.08 to 0.16	0.0 to 0.045
60 kN	0.2 mil.	0.14 to 0.22	0.05 to 0.13	0.0 to 0.055
60 kN	0.34 mil.	0.21 to 0.29	0.03 to 0.09	0.0 to 0.065
80 kN	0.34 mil.	0.21 to 0.35	0.09 to 0.15	0.045 to 0.090
80 kN	0.55 mil.	0.28 to 0.43	0.01 to 0.09	0.060 to 0.145

Analyses of field measurements

Many researchers have attempted to establish the relationship between number of load applications to failure and the ratio of maximum stress in the pavement to the strength of the pavement as represented by the tensile or bending strength of the concrete. Some of the results are plotted in Figure 5. A general equation that depicts the performance curve can be written as:

$$N = a\left(\frac{Stress}{Strength}\right)^{-b} \qquad (1)$$

where N = number of loads to structural failure
Stress = maximum stress in the pavement
Strength = strength of the concrete
a = damage constant

-b = damage coefficient

Both the RISC (Hilsdorf and Kesler 1966) and ARE (Treybig et al. 1977) curves are based on AASHTO data, the first assuming a terminal serviceability index of 2.0 and the second (ARE) a terminal serviceability of 2.5. Referring to *a* and *b* in equation 1 above, the value of *b* for the RISC data is 4.3 and that for ARE 3.2. The value for *b* of the RSA curve also is 4.3, but the difference between the RSA and RISC curves is the value of *a*. Darter (1977) carried out laboratory beam fatigue tests to produce the curve in Figure 5 which is similar to the findings of other laboratory studies. The performance curve used by PCA (Portland Cement Association 1984) is also illustrated in this figure and is used, in similar format, for design procedures in Australia and Canada.

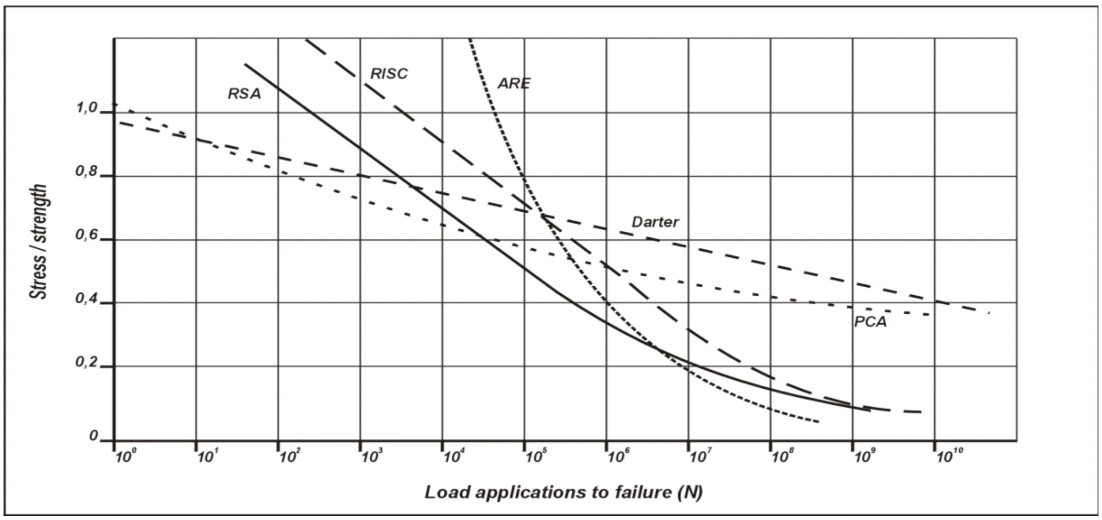

Figure 5. Load applications to failure as a function of the maximum stress in the pavement and the bending strength of the concrete.

The curves defined as RISC and ARE in Figure 5 were all developed from field data as opposed to the Darter curve which was based on laboratory studies. Initially the program cncRisk (Strauss et al. 2001) was based on the RSA curve, but with data subsequently obtained from concrete pavements presently under traffic in South Africa, a value for *b* closer to 4.5 was found. This implies a curve with a slope slightly flatter than the RSA curve indicated above and closer to the PCA curve which has a *b* value of 15.

The performance of the CRC inlay provided an opportunity to evaluate and calibrate the performance equations presently being used in cncPave. Testing of the inlay at a cluster of transverse cracks using the HVS, has indicated that the life or load carrying capacity of the pavement is significantly higher than that for which it was originally designed. However, a survey of the structural condition of the pavement shown in Table 1, showed some deficiencies that needed to be addressed. The survey, carried out in 2004 some 6 years after construction, indicated that about 60% of the punch-outs had occurred at, or close to, transverse construction joints. The majority of these had occurred within two years of construction and it was found that these could be associated with concrete of variable or dubious quality.

Table 3 shows some concrete strength results obtained from an area where punch-outs had occurred. The 28-day cube strengths were obtained at the time of construction and the fresh concrete for the tests was sourced from the mixing plant. The cores were randomly taken where punch-outs were occurring in the pavement, and the concrete at the time of coring was approximately 450 days old.

Table 3 Core strengths (MPa) as a percentage of 28-day cube strengths (MPa) where punch-outs had occurred

28-day Cube strengths		450-day Core strengths		Core strength as % of cube strength	
Average	Standard dev.	Average	Standard dev.	Average	Standard dev.
40.67	1.81	37.05	6.21	90.35	12.35

The effect of variable concrete strength on the development of punch-outs can be illustrated by a plot, shown in Figure 6, of variation in concrete strength versus percentage of inlay failed after 7 million equivalent load applications. The mechanistically-based design program cncPave, which uses variation in input parameters to predict the area failed through Monte-Carlo simulations, (Strauss et al. 2001 and 2004) was used to develop the curve. The area failed was calculated assuming an absolute maximum value for concrete strength likely to be attained in the field and the coefficient of variation for concrete strength was allowed to increase from 0 to a value of 0.17. Figure 6 clearly illustrates the importance of uniform concrete in the construction of inlays. It is also clear that a coefficient of variance in concrete strength above 0.15 implies a high risk of punch-outs occurring.

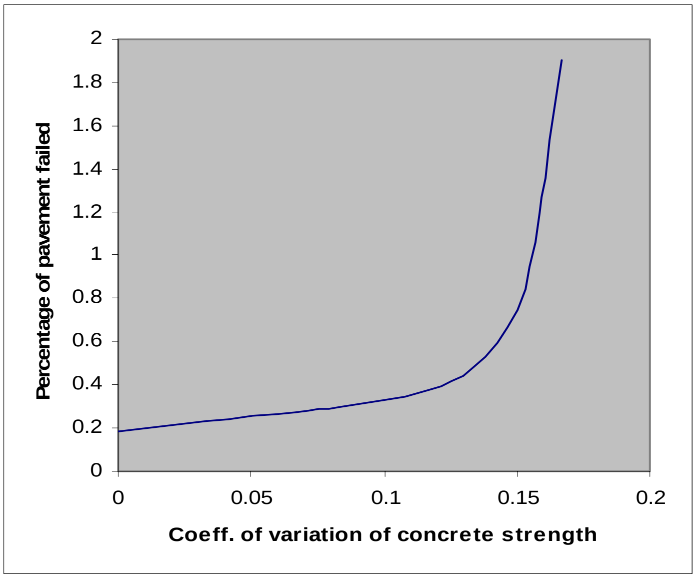

Figure 6 The effect of a variation in concrete strength on the development of punch-outs

Modelling the CRC inlay using the program cncPave and using the as-built information obtained from the construction report issued by South African National Roads Agency Ltd (1998), the present condition of an average of 0.4% punch-outs is simulated if 35% edge loading is assumed, the damage constant "*a*" in equation 1 is taken as 500 and that of coefficient "*b*" as 4.5. A value of 500 implies that 4m^2 of potentially failed concrete around each of the punch-outs is removed and replaced.

Table 4 presents the effect of changing the value of constant "*a*" in equation 1 on the area of concrete to be replaced where a punch-out is to be repaired.

Table 4. The relationship between constant "*a*" and the area around a punch-out to be replaced

Constant "*a*" (eq.1)	Area to be replaced
500	4m^2
1000	2m^2
2000	1m^2

Based on the visual observations and testing results shown in table 3, it is clear that 60% of the failures that occurred during the first 3 years after construction were as a result of poor construction. If all the areas that have a potential for punch-outs now, (a value of 0.4% in table 1), are to fail within the next four to five years, 0.8% of the area of concrete pavement will show structural failures by 2009.

Figure 7 shows a plot that can be compiled using this information as well as a more detailed evaluation using the concrete pavement design program cncPave. This figure shows the best estimate of the area of failure likely to occur (defined by the mean on the graph) as well as the optimistic (ninetieth percentile) and pessimistic (tenth percentile) values based on variation in design parameters.

Using actual performance and modelling, this predicted development of punch-outs with time renders the following equation:

$$\text{Percentage of pavement to be repaired} = 0.0018\,(110\, n^{0.4} + 0.01\, n^{4.0}) \qquad (2)$$

where n = number of equivalent load applications

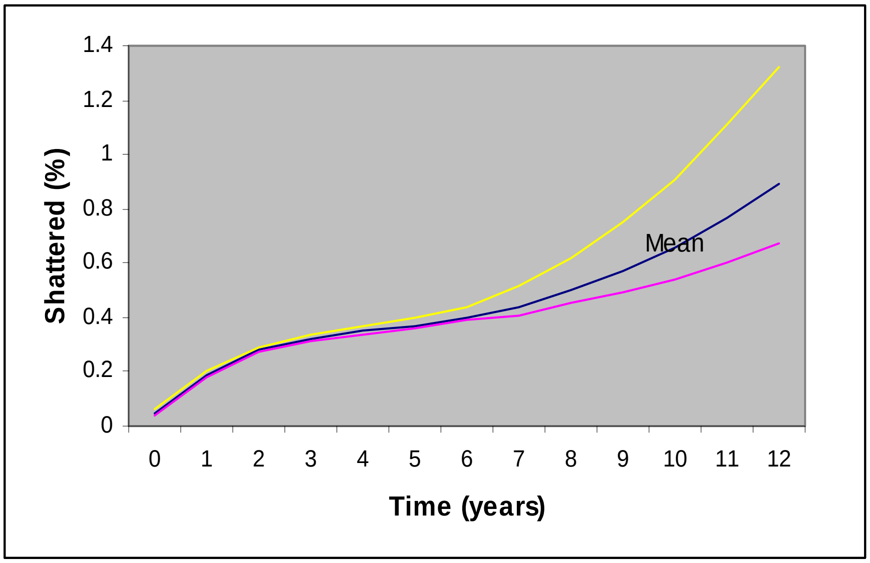

Figure 7. Percentage cracked CRCP on N3 to be repaired as a function of time

Implementation

The CRCP inlay on the N3 was constructed some six years ago and it had already carried its design traffic loading of 6 million equivalent 80kN wheel loads. The use of the HVS on a section of the inlay on N3 where structural failure seemed imminent was an ideal opportunity to refine the performance model and to establish the remaining life, and thus the performance, of such inlays. In this way a better understanding of the link between HVS performance predictions and actual performance under real traffic could also be created.

Due to a lack of funding and limited space for testing, the testing of the inlay was carried out in one position only. Unfortunately this restriction increased the risk of testing a section that, although imminent failure seemed to be present because of the crack pattern, still had significant remaining life.

Although HVS testing was done at a cluster of transverse cracks where structural failure in the form of punch-outs was expected, a punch-out could not be created even though high wheel loading was applied and a significant amount of water was introduced into the pavement. This result has indicated that the life of that particular pavement section is significantly higher than designed for and that there is still a significant amount of remaining life in the pavement. A survey in 2004, the theoretical end of the design life of the pavement, showed that 0.4% of the pavement area had already needed to be removed and temporarily patched using asphaltic concrete, where four square meters of concrete had been removed and replaced at each punch-out.

The possible explanations for the difference in the findings from HVS testing and reality include:

- Punch-outs developed primarily on the inside of curves where traffic loading occurred at the edges of the inlay. HVS testing was done on a straight section in the middle of the lane width and not at a free edge.

- The edges of the pavement were not well protected against the ingress of water and this, together with edge loading and resulting higher deflections, resulted in a loss of slab support and a void developing under the slab.

- About 60% of the punch-outs had occurred close to transverse construction joints and within two years after completion of construction. The majority of these failures appeared to be associated with concrete of variable and lower than specified quality or delays in placing as a result of using labour based methods.

- The only suitable site for HVS testing was a straight section of road with a concrete drain added to the shoulder and where edge loading did not occur under normal traffic loading. Although a free edge was created for HVS testing by cutting the ties to the concrete shoulder, water was introduced into the slab support system and loading occurred in the wheel track of traffic, no failures could be effected. It seems fair to assume that, at this point, the pavement had not suffered “damage” under normal traffic before HVS testing to the same extent as the rest of the pavement, and that therefore the true remaining life of this section of inlay was not determined by the HVS.

- The HVS test site was at a position where the quality of the concrete was superior to the quality of the concrete where failures have and continue to occur.

Due to restrictions such as funding, safety, slope of the pavement and the absence of a concrete shoulder, the HVS test was conducted at one site only, rendering insufficient data to determine failure. However, in spite of this deficiency, very useful behavioural data was obtained and was used in the overall evaluation of load transfer at cracks and the contribution of steel reinforcement to load transfer efficiency. This has already been discussed in previous paragraphs.

Conclusions

Although the performance evaluation was based on the results of only one HVS test section, the information from the condition surveys, the as-built data of the CRC inlay and the analyses of the full length of the CRC inlay, yielded the following useful conclusions:

- The values of both the damage coefficient and the damage constant presently used in cncPave are valid. A value of 500 for the damage constant implies that in repairing a punch-out, four square meters of concrete is removed and replaced whilst a value of 1000 implies an area of two square meters of concrete be removed and replaced.

- A significant percentage of punch-outs occurred where the quality of concrete was substandard and/or where edge loading occurred. These should be avoided when constructing inlays.

- The introduction of water into the supporting layers of the slab increases the risk of failure because of a loss of bond between the slab and the subbase and a void developing in this area.

- The high percentage of longitudinal steel in a CRC inlay contributes significantly to its performance. A lack of sufficient transverse steel in a CRC inlay implies that it should rather be considered as a jointed pavement in the transverse direction. Longitudinal cracking may develop where edge loading and pumping occur together with the development of voids under the edge of the pavement.

- Punch-outs initially occurred in the CRC inlay within the first two to three years after construction as a result of a combination of the above-mentioned factors. These have been estimated to amount to about 60% of the 0.4% of the area of inlay that experienced punch-outs and which had to be repaired by removal and replacement of damaged concrete. A further 0.4% of the pavement shows some distress that may develop into punch-outs in future and will have to be repaired.

- Considering the above conclusions, and based on performance modelling and analyses, it was found that once the first 0.4% of pavement area has been repaired, a further 0.2% to 0.9% (with an average of 0.4%) of the pavement area will have to be repaired by 2008. This implies that if the existing punch-outs are correctly repaired now, the CRC inlay should be in the same structural condition in 2008 as it is now before the repairs.

- The development of structural failure of CRCP with traffic loading has been found to follow an S-curve, with a high incidence of failure occurring initially followed by a stable period before the rate of failure increases again. This trend needs to be confirmed for other types of pavements.

Finally, the information gleaned from HVS testing, together with the testing of adjoining sections, has provided useful insight into the structural performance of concrete pavements. This information, together with the models developed, will be incorporated in the design procedures and specifically in the upgrading of the cncPave design program.

Practical lessons learnt

The two most important lessons learnt were:

- More vigilance is required in terms of concrete quality, consistence and delays at commencement of paving each day – especially if construction is labour based

- Narrow crawler/climbing lanes invariably lead to excessive edge loading and greater risk of failures. "Geometric" (lane configuration to accommodate a wider slab) improvement should therefore be seriously investigated to reduce this risk

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