# HEAVY VEHICLE SIMULATOR (HVS) EVALUATION OF LOAD TRANSFER EFFICIENCY AND CONTINUOUSLY REINFORCED CONCRETE INLAYS ON THE N3 NEAR PIETERMARITZBURG

**L du Plessis[1], P J Strauss[2], B D Perrie[3], and D Rossmann[4]**

[1]Research Group Leader: Accelerated Pavement Testing, CSIR Built Environment, P.O. Box 395, Pretoria, 0001, South Africa
[2]Consultant, P.O. Box 588, La Montagne, 0184, South Africa
[3]Technical Manager, Cement and Concrete Institute, P.O. Box 168, Halfway House, 1685, South Africa
[4]Pavement and Materials Specialist, South African National Roads Agency Ltd, P.O. Box 100 410, Scottsville, 3209, South Africa

## ABSTRACT

The paper addresses two different HVS studies conducted on concrete: (1) load transfer through aggregate interlock and the use of dowels, and (2) the evaluation of the performance of an in-service continuously reinforced concrete inlay on National Route 3 near Pietermaritzburg.

It is well known that the performance of plain jointed concrete pavements depends on aggregate interlock to transfer loads from one slab to the next. In order to quantify the relative contribution of crack width and the strength of the aggregate to the long-term performance of a plain jointed pavement, experimental sections were constructed using different aggregates types. These sections were subsequently loaded to failure using the Heavy Vehicle Simulator (HVS).

The prediction of crack width using the RILEM model which predicts early age shrinkage is discussed in the paper. The model was modified to include the effects of aggregate type, environmental condition and age. The change in load transfer at the joints and cracks, as indicated by relative vertical movement under dynamic loading as a result of temperature variation and humidity, is reported on. It was found that a change in load transfer occurred under increased loading and that this could be related to the crushing characteristics of the coarse aggregate.

The paper presents the final outcome of the study in terms of theoretically based equations that were adjusted using regression techniques to fit the field experience. These equations have now been incorporated into a mechanistically-based design method for concrete pavements, namely cncPave.

For the second HVS study, the residual life of an existing CRCP inlay was determined. The asphalt on the slow lane on a steep uphill section of the National Route 3 near Pietermaritzburg was milled and replaced with a 180mm of CRCP inlay in 1998. 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 subsequently 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.

In the paper, the behaviour and performance of a relatively highly cracked section of the inlay subjected to Heavy Vehicle Simulator (HVS) trafficking are discussed. The results were translated into transfer functions which now also have been integrated in cncPave.

## 1. EVALUATION OF LOAD TRANSFER EFFICIENCY

### 1.1 Background

Concrete pavements have been designed and constructed in South Africa using “modern” technology since the 1960s. The performance of several of these sections has and is still being monitored and the information is being used in upgrading design and construction methods. Incorporated into some of the above sections were short test sections of thin concrete pavements that have been intensively tested, including trafficking with the Heavy Vehicle Simulator (HVS), and monitored with time.

As a result of the need to develop a mechanistically-based design method for concrete pavements, an overall plan was drawn up to address necessary aspects of the design process and included a motivation for the revision of the nomogram-based M10 Design Manual. Subsequently, a new mechanistically- and computer-based design method, cncPave, was developed (Strauss et al. 2001 and 2004). Following more inputs from research and the performance of test sections and real road sections, the program has been further refined to predict the extent rather than the risk of failure.

The program now consists of modules that address the following aspects:

- External loading as defined by the distribution of typical vehicles;
- The transfer of loads from one slab to the next through aggregate interlock or dowels;
- Slab support stiffness and the loss of this support through erosion, pumping or settlement;
- Slab characteristics including strength and stiffness;
- Stress in the slab as a function of the above variables;
- The structural performance of the slab as a result of stress within the slab;
- Variability of the input parameters and the prediction of the extent of failures with time, and
- Cost implications of the final design.

### 1.2 Research Needs

The design program cncPave is presently used extensively by designers and road owners and has created an awareness of the sensitivity of the different input parameters. Furthermore, monitoring of the program’s reliability together with feedback from practitioners has indicated that the program could be further improved upon, particularly by refinement of the load transfer module.

A load transfer coefficient C was introduced into cncPave to distinguish between the load transfer capabilities of aggregate interlock and dowels in joints so that the differences in performance of Plain Jointed Pavements (PJP), Dowel Jointed Pavements (DJP) and Continuously Reinforced Concrete Pavements (CRCP) could be mechanistically explained. This approach, together with feedback on the performance of concrete pavements in South Africa, is being considered in the process of updating and improving design and construction procedures.

Based on the use of cncPave and from monitoring the performance of existing concrete roads and test sections, it became clear that research was needed to establish the effects of loading on the change in slab support, and the load transfer at joints and cracks.

## 1.3 Theoretical Background

For the purpose of this paper the structural performance of a concrete pavement is evaluated as a function of the maximum stress at a joint or crack in the pavement (Strauss et al 2001):

$$\text{Stress} = f\left(C, \frac{P}{h^2}, \sqrt[4]{\frac{D}{k}}\right) \qquad (1)$$

where: Stress = maximum tensile stress close to a joint or crack in the pavement
C = coefficient that depends on load transfer at a crack or joint
D = slab stiffness
k = slab support stiffness
P = magnitude of load
h = slab thickness.

The magnitude of the load transfer coefficient C is dependent on the aggregate interlock or the dowel action of longitudinal steel bars at the joint or crack. In both cases, load transfer is a function of the relative vertical movement, $\Delta y$, at the joint or crack under a moving load. The slab support is dependent not only on the stiffness of the supporting layer but also on any void that may develop below the slab as a result of slab curling or erosion and pumping.

Based on work by Walraven (1981) as well as a laboratory study to develop the South African Concrete Pavement Design Manual (1990), it was confirmed that relative vertical movement, $\Delta y$, and thus aggregate interlock, is a function of crack width, aggregate shape and size, as well as the strength of the aggregate itself. Relative vertical movement at a joint/crack under the influence of aggregate interlock can be written as (Brink 2003):

$$\Delta y = 0.118(1 - e^{- ((v + 11.4/agg)\Delta x)^{1.881}}) \qquad (2)$$

where $\Delta y$ = relative vertical movement at joint/crack
v = factor influenced by speed of loading
$\Delta x$ = crack/joint width
agg = nominal size of the 20% biggest particles in the concrete mix

In the case of dowel action of steel bars in the pavement, the strength of concrete around the steel and the size of the steel bars are important to reduce or maintain a low level of relative vertical movement at a joint. Relative vertical movement at a joint or crack in which steel bars are installed can be written as (Yoder and Witczak 1975):

$$\Delta y = P\,(2 + \beta x) / (4\beta^3 EI) \qquad (3)$$

where $\beta = [Kb / 4EI]^{0.25}$
K = Winkler stiffness of the concrete around the steel bar
b = steel bar diameter
E = modulus of elasticity of the steel bar
I = moment of inertia of the steel bar
P = load on the steel bar
x = crack width

It is clear from equations 2 and 3 above, that crack width is important in predicting relative vertical movement and thus the successful transfer of load at a crack or joint in the pavement.

Crack width in turn depends on the shrinkage and thermal characteristics of the concrete used in constructing the concrete pavement. Shrinkage can be measured at the time of construction or it can be calculated from other known variables.

The shrinkage strain in concrete can be calculated using the RILEM equation (RILEM 1995):

Strain with time = $S(t)\ k_h\ \varepsilon$ (4)

where: $S(t) = \tanh \{(t-t_0)/4.9D^2\}^{0.5}$ (5)

$\varepsilon = \alpha_1\ \alpha_2\ [0.019\ w^{2.1}/f^{\ 0.28} + 270]$ (6)

$k_h = 1 - h_u^3$

$t$ = age of the concrete

$t_0$ = age when drying starts

$D$ = effective cross section thickness = 2 v/s

(v/s is the volume to surface ratio)

$\alpha_1$ = cement type

$\alpha_2$ = factor for curing

$w$ = water content of the concrete

$f$ = cylinder compressive strength of the concrete

$h_u$ = factor for relative humidity (where 100% humidity = factor of 1)

Adding a factor $\alpha_3$ to account for the influence of different types of aggregate on shrinkage, as suggested by Badenhorst (2003), as well as strain due to the change in the temperature from the temperature at the time of placing the concrete, results in the following equation:

Strain = $C_1\ \alpha_1\ \alpha_2\ \alpha_3/h\ [0.019\ w^{2.1}/f^{\ 0.28}+270]+(T_0 - T_t).\eta$ (7)

where $\alpha_3$ = aggregate type

$C_1$ = constant

$h$ = slab thickness

$T_0$ and $T_t$ = temperature at time of paving and present temperature respectively

$\eta$ = thermal coefficient of the concrete

1.4 HVS Testing

In order to address some of the research needs listed above, a series of HVS tests were conducted on short sections of jointed concrete pavement (JCP) at Hilton as well as on a CRCP inlay at Town Hill N3. Limited testing was also carried out on selected sections of pavement on the N3-3, (the Pietermaritzburg bypass), and the N2-26 & 27 (between Durban and Stanger). To gain full value from these tests it was important that each test should be conducted in a systematic way so that the performance of the variable under investigation could be evaluated. In this way a substantial database of all possible variables that have an influence on pavement behaviour was created. However, since the main aim of the study was to improve the models in cncPave, the following were tested as part of this study and will be reported on in greater detail in this paper:

- *The change in load transfer with traffic loading and time.* Load transfer is a function of aggregate interlock and the dowel action of steel bars, when used. Aggregate interlock is a function of crack/joint width, aggregate shape and size as well as the strength of the aggregate itself. In the case of dowel action of steel bars in the pavement, the strength of concrete around the steel and the size of the steel bars are important. However, the wear or abrasion of aggregate as well as the concrete with loading and time had to be established. Dolerite and quartzite aggregate was used in the two sections at Hilton that were tested with the HVS to determine the influence of the hardness and type of aggregate and its performance under loading.

- *Prediction of thermal and shrinkage behaviour of concrete in a pavement.* Load transfer at a joint or crack is a function of the width of the crack or joint. This in turn is directly dependent on the shrinkage and thermal movement of the concrete used in the pavement. The test sections at Hilton provided an excellent opportunity to investigate these factors and to supplement the data with actual performance of pavements on the N3 and N2.

The scope of the accelerated pavement-testing programme was limited to the investigation of doweled and plain jointed 3.5m x 4.0m x 150mm thick slabs with both 19mm dolerite and quartzite stone. In order to evaluate the relative contribution of these variables to joint behaviour and thus to performance of the pavements, the test sections were constructed on top of three 150mm layers of natural gravel with in situ CBR of 15 (Brink 2004). The test site is shown in Figure 1.

Details of the as-built properties of the sections are contained in the construction report (Brink 2004).


**Figure 1. General view of the test site.**

Each test section was subjected to 40kN wheel loading using unidirectional load application. The HVS is capable of applying 10 000 unidirectional wheel loads per day. A relatively low strength slab support was selected and water was introduced with a drip irrigation system through small holes in the pavement into the interface between slab and support to induce failure of load transfer without slab failure itself. Vertical and horizontal movements at the joint were monitored and environmental data as well as the variation in crack width was measured on a regular basis,

Figure 2 shows the typical crack pattern that had developed by the end of the HVS test. The transverse joint is indicated in a reddish colour while the cracks that developed later are indicated in yellow and blue, the blue being a crack that developed at the very beginning of the testing. The crack pattern indicates a bigger void under the leave slab than under the approach slab. This void was as a result of pumping (as shown in Figure 3), where pumped material is more prominent at the side of the leave slab.


**Figure 2. Crack pattern at the end of the test on the undowelled dolerite section.**

**Figure 3. Pumping from under the leave slab, the right hand side of the crack.**

**Figure 4. Spalling of a crack as a result of relative vertical movement and pumping.**

As a result of the void and thus larger relative vertical movements at the joint and the cracks, spalling as shown in Figure 4 occurred. For greater detail on the sequence of distress at each test, reference is made to the first-level-analysis report (Brink and du Plessis, 2004).

1.5 Analyses of Field Measurements

The main purpose of this paper is to report in general on the refinement of the design methods for concrete pavements and on the cncPave program in particular. The analyses are focused on the effect of an increase in the number of loads on joint behaviour and performance. Data generated by HVS and other testing was used for this purpose and analysis of data was carried out using statistical principles. However, pure regression analyses may be misleading since the reliability of the resulting equations will depend on the reliability of the data. The study is based on limited testing and in order to extend the applicability of the equations generated to beyond the experimental data, equations needed, wherever possible, to be based on theoretically derived relationships.

Test sections were well instrumented on both the surface of the pavement at and between joints and cracks, and as well into the depth of the pavement. The analyses of data obtained from these measurements were based on theoretical models already developed and discussed earlier. Data generated by measurements of crack widths at different temperatures, FWD deflections, and pavement behaviour with increased HVS loading, were used for this purpose. The analyses of data were done using regression techniques but based on theoretical models already discussed above. Although a great number of data points were obtained (especially under HVS testing), the variation in readings was such that a wide spread of results was obtained. Forcing this data into a format obtained from theoretical models, generally resulted in low $R^2$ values, but the benefit of this approach is that the reliability of the resulting equations are high in terms of their applicability beyond the experimental data.

***Crack width.*** The total shrinkage strain in a concrete pavement constitutes two phases – a first phase of shrinkage strain over the first few weeks, assuming controlled curing, and a second phase consisting of the long-term shrinkage that depends primarily on the age of the pavement and the environment under which the pavement is performing.

Shrinkage over the longer term can be calculated using the following equation that was derived using data generated by Troxell (1958):

$$\varepsilon_t = C_2 [900\text{-}t] [t\text{-}0.08]^{0.18} [1\text{-} h_u] \qquad (8)$$

where: $h_u$ = relative humidity (value of 1 = 100% humidity)
$t$ = time (years), and $C_2$ = constant

Crack width Δx can be calculated by combining equations 7 and 8:

$$\Delta x = [C_3/h\{\alpha_1\, \alpha_2\, \alpha_3\, (0.019w^{2.1}/f^{\,0.28}+270) +(900\text{-}t)\, (t\text{-}0.08)^{0.18}\}(1\text{-}h_u)+(T_0–T_t)\, \eta].L \quad (9)$$

Crack widths on the surface of the pavement were measured daily at 8h00 and 14h00 using a microscope. Air and concrete surface temperatures as well as humidity were recorded at the same time. Crack widths were subsequently also measured inside cored holes for all joints and cracks on the Hilton sections as well as on some sections of road on the N3 and N2. It was found that the crack widths measured on the surface of the pavement using the microscope were between 0.20 and 0.32mm wider than the core measured values.

The data was subsequently used to compare measured crack widths with theoretically predicted values determined using equation 9. In the case of the data from the N2 and N3, the average daily values of humidity and the concrete characteristics at the time of construction were obtained from design mix data. Despite the lower accuracy of the environmental and concrete mix data thus obtained, an $R^2$ of 0.61 was still arrived at for equation 9. It was interesting to note that air temperature rendered a better correlation to crack movement than did concrete surface temperatures. Since the air temperature can be predicted more easily in the design process, it was decided to use it rather than concrete surface temperature in the finally derived equation. Figure 5 shows a plot of measured versus calculated crack widths obtained from equation 9 and using core hole measured crack widths and air temperature as variables.

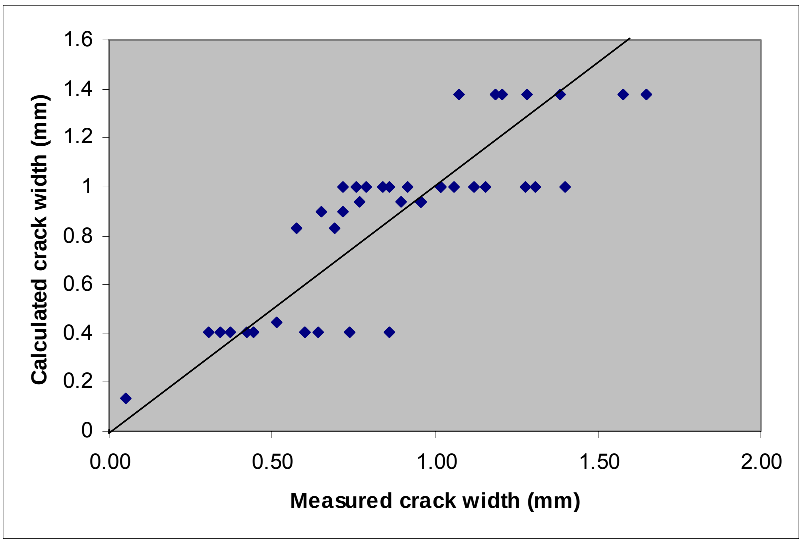


**Figure 5. Measured versus Predicted Crack Widths.**

***Relative vertical movement.*** Relative vertical movements were measured under both HVS and FWD loading. The HVS testing was conducted using unidirectional travel and a 40-kN wheel load. Movement at cracks was measured by using multi-depth deflectometers (MDDs) as well as joint movement deflection measuring devices (JDMDs). MDDs are anchored about two meters below the surface thus measuring the absolute movement at different levels in the pavement structure while JDMDs are installed on the surface and measure only surface movements relative to the anchor. Both these instruments were found to be accurate in measuring relative vertical movement under the rolling wheel load and a high correlation was found between the two measuring instruments.

The following equations were generated from regression techniques using 1475 data sets obtained from HVS testing:

$$\Delta y = 8.37\ (\Delta x_m{}^{1.5} / Agg) + 0.030.n.\ ACV - 0.254 \qquad (10)$$

$$\Delta y = 2.22\ (\Delta x_c{}^{1.5} / Agg) + 0.030.n.\ ACV - 0.040 \qquad (11)$$

where: $\Delta y$ = relative vertical movement (mm)

$\Delta x_m$ = crack width using actually measured values in equation 10 (mm)
$\Delta x_c$ = calculated crack width using equation 9 to calculate crack width (mm)
Agg = nominal size of the 20% biggest particles in the concrete mix (mm)
n = number of load applications actually applied (million E 80's)
ACV = aggregate crushing value, an indication of the strength of the aggregate (larger numbers indicate weaker aggregate)

and $R^2$ values of 0.57 for equation 10 and 0.37 for equation 11 and Standard Error of the Estimate (SEE) of 0.12 and 0.14 respectively.

The calculated crack widths do not render the same reliable relative vertical movement at the crack under HVS loading (equation 11) as do the measured crack widths (equation 10). Note that the slab thickness, the humidity and the rainfall were not significant (did not contribute to an improved $R^2$) or the coefficients did not make engineering sense. Accepting that equation 10 depicts the best relationship between relative vertical movement and crack width, a plot of the measured relative vertical movement versus the predicted values can be compiled and is shown in Figure 6 below.

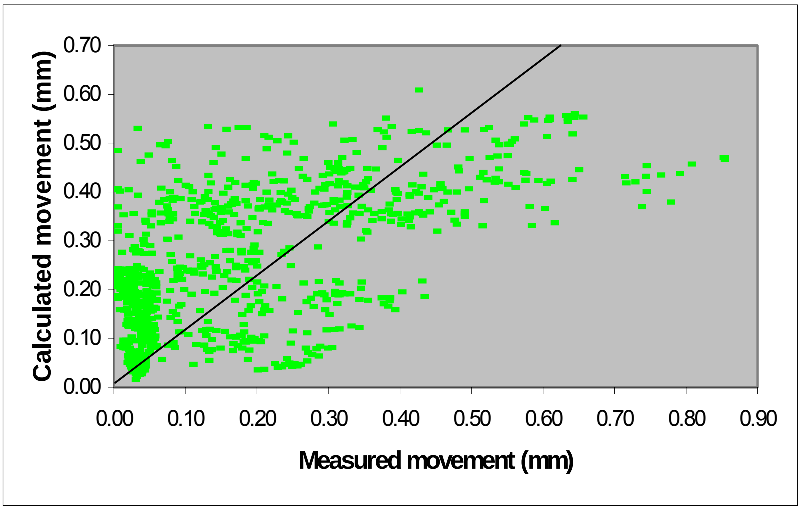


**Figure 6. Calculated versus Measured Relative Vertical Movements under 40 kN HVS loading.**

Relative vertical movement was measured using the FWD equipment on the sections tested by the HVS as well as selected sections on N2 and N3. The loading plate of the FWD was placed on the leave side (down stream of the joint as traffic moves) of the joint and deflections were measured on either side of the joint. It was found that calculated crack widths, traffic loading, strength of the aggregate and bond between slab and subbase showed the following relationship:

$$\Delta y = 7.4 \, [\Delta x_c^{1.5} / Agg\,] + 0.0015 \, .n \, (ACV - 2.5 \, Bond) \quad (12)$$

where: $\Delta x_c$ = crack widths using equation 9 to calculate crack width
Bond = bond between concrete and subbase.

Based on 50 data sets the $R^2$ = 0.67 and SEE = 0.076 for this equation

Using equation 12 to calculate relative vertical movements and comparing them to the measured relative vertical movements renders the plot as shown in Figure 7 below.

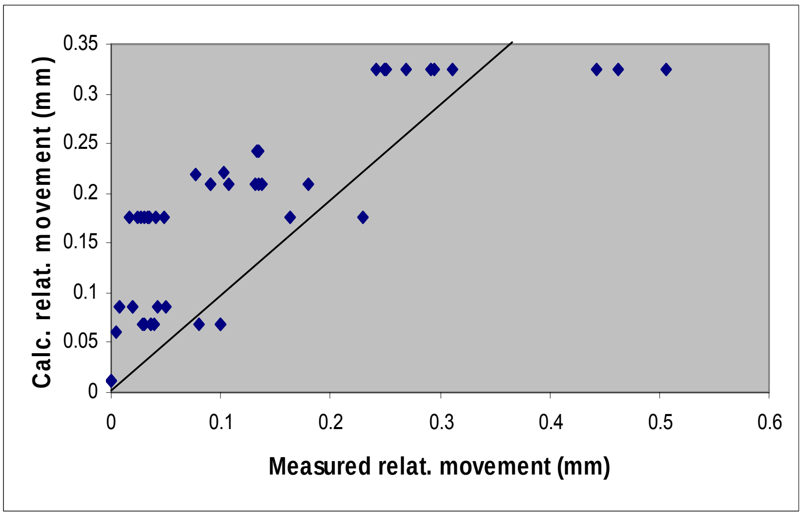


**Figure 7. Calculated versus Measured Relative Vertical Movements under FWD loading.**

In equation 12, bond between slab and support is defined subjectively on a scale of 0 to 10 where 0 implies no bond at all and 10 a perfect bond. In this study it was found that, in cores drilled from the N3, where the slab had been placed on asphalt, the concrete had bonded much better than on the cemented subbase of the N2. As a result, a value of 8 was assigned for bonding on the asphalt and a value of 2 for the cemented subbase. It is interesting to note that the stiffness of the subbase played a lesser role than the bond between subbase and slab based on the outcome of the regression analysis using the data in the database.

***Aggregate interlock.*** Aggregate interlock is also a function of the roughness of the concrete surface inside a crack. In an attempt to quantify this roughness, the volumetric surface texture ratio (VSTR) was determined by measuring the cracked surface using laser technology and dividing it by the flat area of the sample (Brink 2003). The VSTR of the crack after being trafficked was compared to the sections that had not carried any traffic and was found to be less textured, but the difference was insignificant when different samples were compared.

***Dowel action.*** Steel reinforcement in a CRC or dowels in a dowel-jointed pavement contribute significantly to load transfer at a joint or crack. The basic equation to quantify this phenomenon has been discussed previously and is reflected in equation 3.

Simplification of equation 3 together with regression analyses on data obtained from the HVS, both from the short trial sections at Hilton as well as the CRC inlay on N3-3, resulted in the following equation for the dowel action of steel:

$$\Delta y = 0.4 \, Spac \, P^{2.0} \, n^{0.16} / (dia^{1.75} \, E^{0.75}) \quad (13)$$

where: Spac = spacing of steel bars (m)
P = wheel load (kN)
n = number of wheel load applications (million)
dia = diameter of steel bar (mm)
E = stiffness modulus of the concrete surrounding the steel (MPa)
Based on 2223 data sets, the $R^2 = 0.48$ and the SEE = 0.023 for this equation.

The basic format of equation 13 is similar to the theoretical equation 3 and the effect of the number of load cycles was statistically determined. Because of this approach, the value of $R^2$ is relatively low, but the resultant variable $n^{0.16}$ indicates that the number of load cycles needs to be considered when predicting a change in load transfer capability of dowels or steel in the pavement.

<u>1.6 Implementation</u>
The original aim of the study was to establish the effect of number of load applications on the deterioration of load transfer at a joint or crack and on the loss of slab support.

All the data from the Hilton experiment and the N3 and N2 were evaluated and used in statistical analyses to enhance the theoretical equations developed for load transfer at joints and cracks, erosion of the slab support system and prediction of the performance of concrete pavements. The more significant findings that can be derived from the study as well as their application in practice, particularly in strengthening the design package cncPave, are the following:

***Shrinkage.*** Shrinkage can be measured under controlled conditions and on small samples in the laboratory using recognized standard methods. However the relevance of accelerated laboratory test results in predicting shrinkage in a pavement is questioned (Badenhorst 2003). An equation was developed to calculate the initial shrinkage using fundamental properties such as water content, cement type, aggregate type, 28-day compressive strength and type of curing. Further shrinkage depends primarily on the age of the concrete and the humidity of the environment in which the slab is performing. The width of the crack in a pavement can then be calculated by combining both shrinkage and thermal behaviour. The calculated crack width was compared with the measured crack width and a final equation was derived through regression analyses. In compiling the equation, it was found that air temperature was a marginally more reliable predictor of thermal movement than concrete surface temperature. Air temperature was therefore used in the remainder of the analyses because of the relative ease of measurement.

***Crack width.*** Measuring the crack width has proven to be a difficult operation with variation in results depending on the method of measurement and the position of measurement, whether on the surface or within the slab. Eventually feeler gauges were used to measure the crack width inside a core hole. Because of the difficulties in accurately measuring crack width, the reliability of the predictions (R2) was not high. The resulting regression equation 9 is therefore based on measuring crack width using a feeler gauge, and the values obtained from using the equation should be interpreted as such.

Figure 8 shows a plot of calculated crack width with time as a function of the more important variables – temperature and humidity. Typical values of temperature and humidity for Upington, a very dry desert area, and Durban, with a sub tropical climate, were used to compile the figure. The joint spacing was assumed to be 4,5m and a mix normally used for concrete pavements was used. The effects of cement type, water content and compressive strength are less important when compared to temperature, age and humidity, and were therefore kept constant in this case.

Equation 9 has been implemented in the design program cncPave to calculate crack width. Instead of using shrinkage values of mixes determined in the laboratory, the accuracy of which is now being questioned, variables such as the water content and the amount of paste as well as the strength of the mix, which are better known to the designer, can be used to predict crack width. Furthermore the change in both temperature and humidity with time can now also be introduced to increase the reliability of the predictions of pavement behaviour.

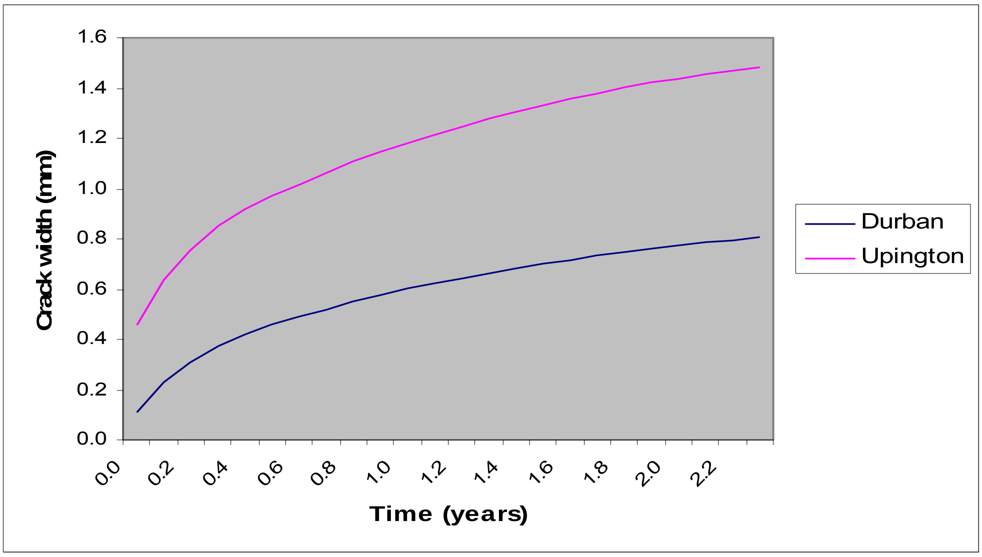


**Figure 8. Crack width as a function of time for different climates.**

***Aggregate Interlock.*** Load transfer is a function of aggregate interlock and the dowel action of steel bars where they are used in the pavement. Aggregate interlock is a function of crack/joint width, aggregate shape and size as well as the strength of the aggregate itself. In the case of dowel action of steel bars in the pavement, the strength of concrete around the steel and the size of the steel bars are important. The wear or abrasion with loading and time has been established in this study both for the aggregate as well as for the concrete around the steel.

Abrasion of the aggregate depends on its Aggregate Crushing Value (ACV) and the number of load applications. It is a linear relationship and the coefficient has been found to differ depending on whether relative vertical movement was measured under HVS loading or the FWD. In all the equations relating relative vertical movement to crack width, number of load applications, ACV and other variables, the $R^2$ values were found to be relatively low. The reasons for this include:

- The equations were forced into a format dictated by the theoretically derived equations discussed earlier

- Variables that are relatively easy to measure were given priority in the regression process. The reason for this being that the outcome of this study needed to be of use in practice and those variables that cannot be measured or estimated easily would only render equations of academic interest.

- Visual observations during the HVS testing raised some questions as to the effect of moisture on joint movement especially when it was observed that a reduction in movement occurred after “slush” from pumping of the subbase seemed to have “frozen up” the joint. This, however, could not be quantified adequately and it was found that in the regression analysis, rainfall did not play a significant role but that humidity was a more important variable. However, it was still not important enough to make a difference to the reliability of the equations. Humidity was however a significant contributor to predicting crack width and as such was included as a variable. Because of a high correlation between humidity and rainfall, and with humidity contributing more significantly to the reliability of the equations, rainfall was excluded.

- The measured and/or calculated crack width had a significant effect on the prediction of relative movement. However the accuracy with which the crack width could be measured was low. Surface measurements using a microscope resulted in wider crack widths than those determined deeper into the slab. The latter were measured using a feeler gauge, but again the success of these measurements depended on the roughness and shape at the point of measurement of the crack and thus the ability to insert the feeler gauge.

- Not only is aggregate size playing a role in load transfer, but the shape of the aggregate particles as well as the roughness inside the crack influence load transfer. The roughness of the concrete inside the crack was determined and expressed as the volumetric surface texture (VST). The VST of the crack after being trafficked was compared to the untrafficked sections and found less textured. However the difference was statistically insignificant and the measured values were not used.

The coefficient for the *Δx/Agg* variable varies from 2.0 to 8.4 for FWD- and HVS-measured vertical movements respectively, with a realistic average value of 8 for calculated crack widths. However the coefficient for the variable *n.ACV* varies significantly for the different equations 10 to 12, from 0.0015 to 0.032. The coefficient was of the order of 0.0015 for the FWD-measured data and 0.032 for the HVS-measured data. The reason for this large variation can only be speculated on:

- The HVS load, which contributes to an increase in relative vertical movement with time, was applied systematically in one position. The loads under real traffic however, wander significantly across the width of the pavement and their contribution to damage is therefore smaller. If it is assumed that 30% of the weigh-in-motion (WIM) estimated E80's on the N3 and N2 crossed the joint where FWD measurements were taken, the coefficient for *n.ACV* increases to 0.007 instead of the 0.0015 obtained in equation 12 indicating the merit of this approach.

- The HVS load applications applied could be determined much more accurately.

- The conversion of the number of loads to E80's using a damage coefficient of 4.2 is not realistic for *n.ACV*

- The number of load applications applied at the HVS test was limited to 0.75 million, but for the real-life traffic of the N3 and N2 it was as high as 12 million. If the latter traffic figure is used in the equations developed from the HVS data, unrealistic values of more than 10mm relative vertical movements are obtained. The use of a limited database, as was the case for HVS testing, to derive an equation from regression analyses can be misleading if the equation is used to predict movements beyond the limits of the database.

In view of the discussion above, it is recommended that equation 12, developed from FWD data, be used in design procedures. Although equation 11, developed from HVS data rendered a very similar equation, equation 12 is preferred because of a higher level of significance, a more realistic coefficient for the influence of traffic and the inclusion of bond between slab and subbase. Figure 9 shows the results of a plot of relative vertical movement as a function of time for a concrete pavement in a dry hot climate.

Equation 12 was used to calculate relative vertical movement at a joint as a function of time, taking into account traffic loading, the ACV of the stone and the bond between the subbase and the slab. The three cases that were considered and which are shown in Figure 9, are the following:

1. a traffic load of 20 million E80's using a stone with an ACV of 25 and with very little bond between subbase and slab;
2. a traffic load of 20 million E80's using a stone with an ACV of 25 and with high bond between subbase and slab.
3. a traffic load of 5 million E80's using a stone with an ACV of 15 and with very little bond between subbase and slab,

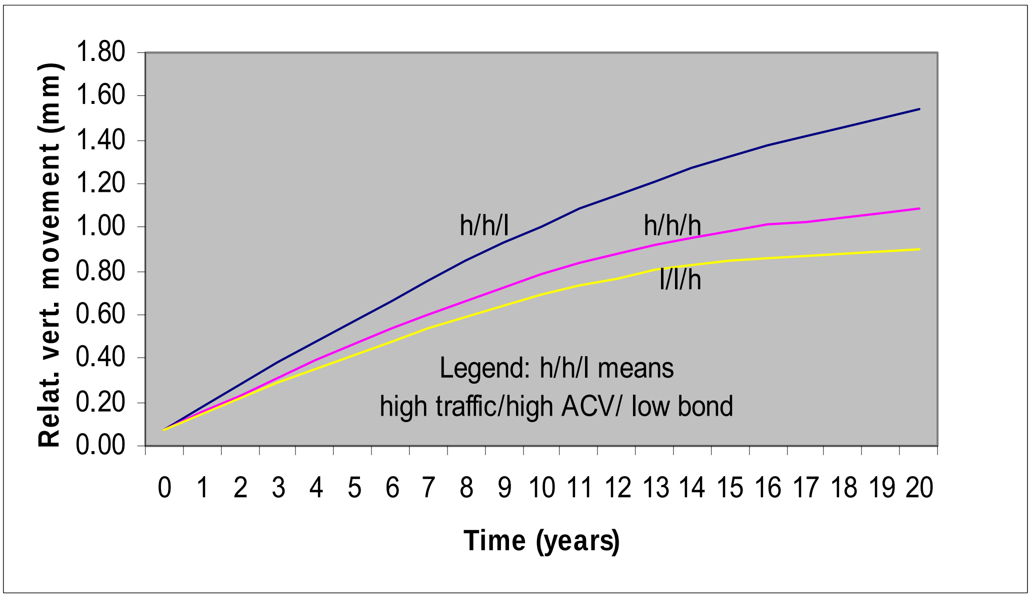


**Figure 9. Calculated relative vertical movement as a function of time, traffic loading, ACV of the aggregate and bond between subbase and slab in a dry, hot climate.**

Important conclusions from this part of the study can be summarized as follows:

- Crack width is an important parameter in the success of load transfer at a crack or joint
- The bonding between subbase and slab was more important than the stiffness of the subbase. This finding should be seen in the context of the limited data available in this study. Theoretically the stiffness of the subbase will play a significant role if high bond exists between these two layers. However, in this study it was found that on the N2 the bond between the cement stabilized subbase and the jointed pavement, particularly in the vicinity of the joints, was limited.

Equation 13 presents the outcome of the change in relative vertical movement with increased loading for a dowelled jointed pavement. Unfortunately the stiffness of the concrete was virtually the same for all the sections and the coefficients for stiffness E in equation 13 came from the theoretical equation 3. Similarly the coefficient for number of loads n has a value of 0.16 instead of being a function of E as would be expected. Replacing 0.16 with a value of 7000/E where concrete stiffness E is in MPa, the equation 13 becomes:

$$\Delta y = 0.4 \text{ Spac } P^{2.0} \, n^{7000/E} \, / \, (dia^{1.75} \, E^{0.75}) \qquad (14)$$

Fig 10 shows a plot of equation 14 with some typical values for concrete stiffness and dowel diameter.

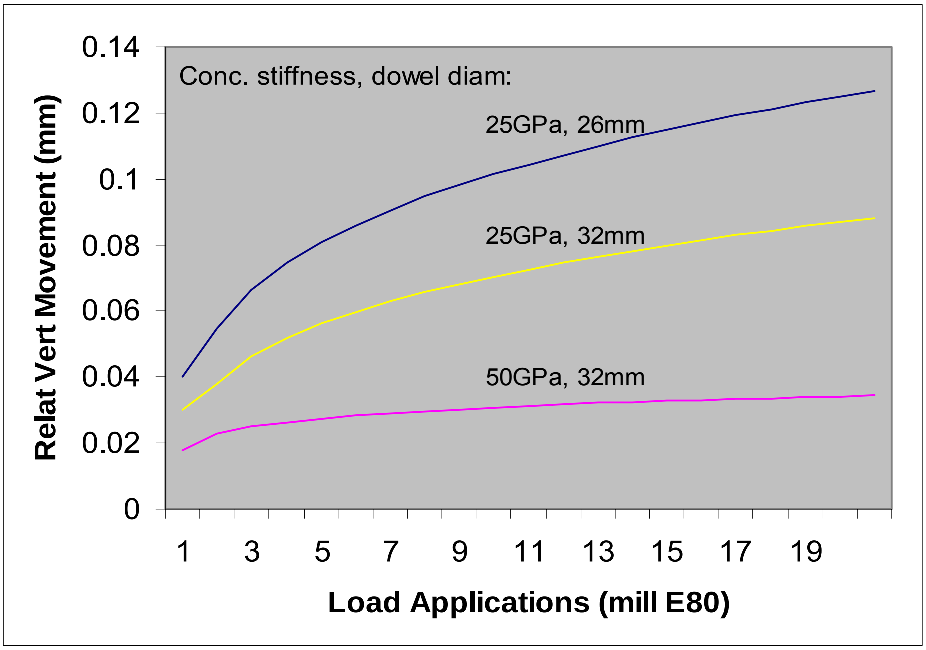


**Figure 10. Relative movement at the joint of a dowelled pavement.**

Figure 10 shows that relative vertical movement (and thus load transfer at a dowelled joint) depends to a great extent on bar diameter, stiffness of the concrete and the number of load applications

## 2. EVALUATION OF THE PERFORMANCE OF AN IN-SERVICE CRCP INLAY

### 2.1 Background

The construction of the 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.

### 2.2 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 11 to 13 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 11 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 12. This is defined as a punch-out since the block will become loose and create a safety hazard. Figure 13 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 11. A cluster of cracks at a transverse construction joint and an open edge joint.**


**Figure 12. A cluster of cracks, an open edge joint and edge loading lead to a punch-out.**

**Figure 13. 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.

2.3 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 14, cracks occur in a cluster with some secondary cracking already occurring – suggesting the possibility of imminent failure.

**Figure 14. Test site for HVS testing.**

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.

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 show 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 |

2.4 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:

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

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 15 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.

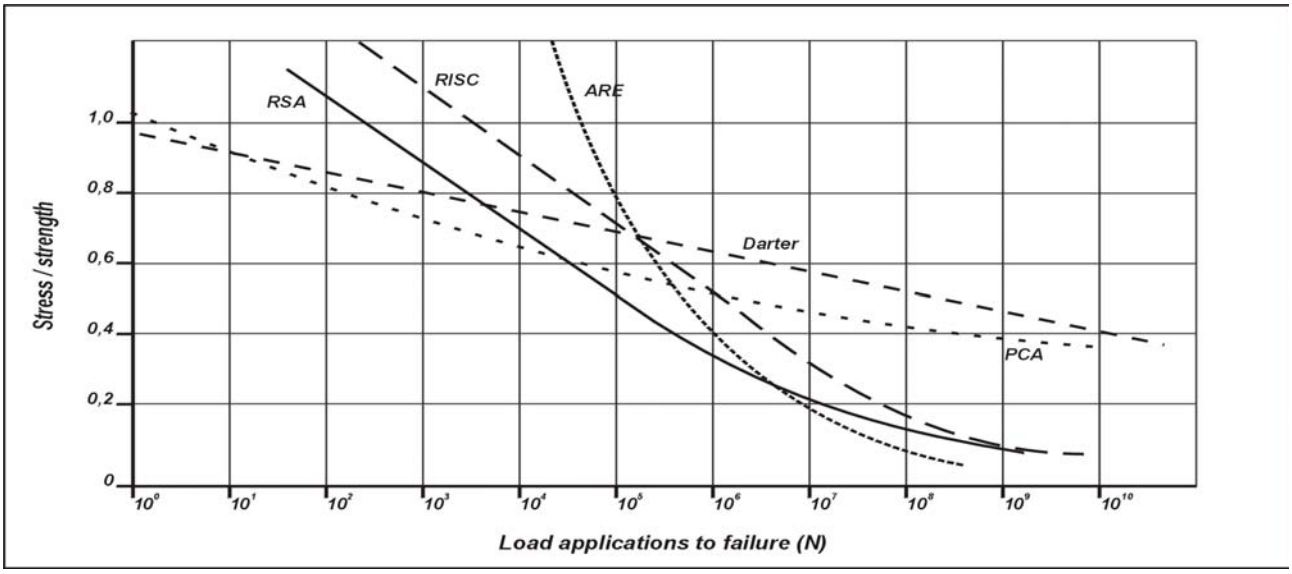


**Figure 15. Load applications to failure as a function of the maximum stress in the pavement and the bending strength of the concrete.**

The curves defined as RISC and ARE in Figure 15 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 | 2.35 |

The effect of variable concrete strength on the development of punch-outs can be illustrated by a plot, shown in Figure 16, 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 16 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.

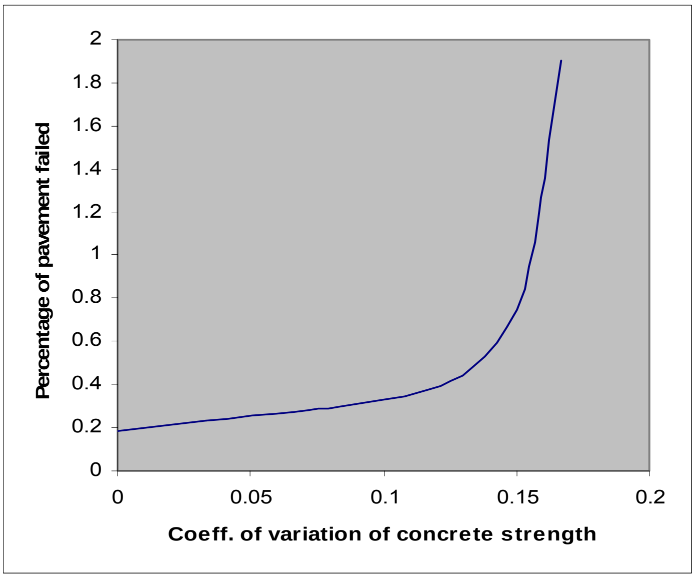


**Figure 16. The effect of a variation in concrete strength on the development of punch-outs.**

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 17 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

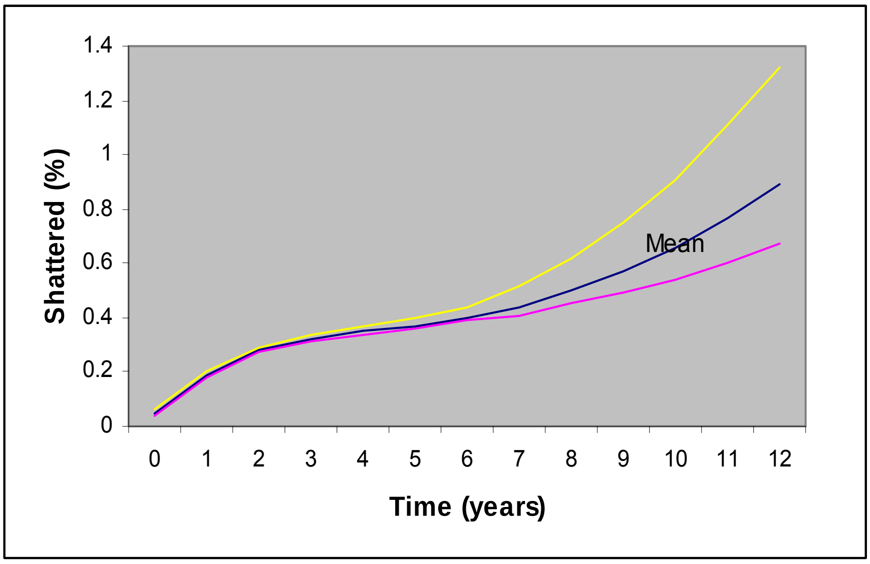


**Figure 17. Percentage cracked CRCP on N3 to be repaired as a function of time.**

<u>2.4 Implementation</u>

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.

## 3. CONCLUSIONS

### 3.1 Evaluation of Load Transfer Efficiency

Short sections of jointed unreinforced concrete pavement were constructed near Hilton, adjacent to the N3, to establish the effect of different aggregate types and dowels on the performance of joints under HVS loading. The most important outcomes of the first phase of HVS testing are briefly summarised below:

- Pavement behaviour was monitored in great detail and an extensive database has been established. However due to environmental (predominantly rainfall and temperature) and other influences outside the control of operators, scatter of data was wide and influenced the reliability of the predictive equations derived from regression analyses

- In order to be able to use the resulting equations in environments outside those experienced on the selected sections, regression equations were forced to follow theoretical formats. This also added to the reduced reliability (using $R^2$).

Despite the above-mentioned constraints, very useful results were obtained – the most important being confirmation that structural failure is associated with poorly performing joints or cracks. The following specific conclusions, pertaining primarily to the load transfer efficiency of joints and cracks, have been arrived at:

- The hardness of aggregate plays an important role in the long term load transfer efficiency of a joint or crack; aggregate with a lower ACV is preferred for better long term performance

- Crack width is however the most important parameter in the load-transfer efficiency of a joint or crack. Crack width is in turn affected by the shrinkage characteristics of the concrete as well as by environmental factors such as the variation in temperature and relative humidity. High concrete shrinkage, substantial temperature variations, and a low relative humidity are detrimental to the performance of jointed concrete pavements

- The bond between the concrete slab and the subbase directly below the slab has a significant influence on the behaviour, and thus the structural performance, of the pavement. High bond was found wherever asphalt was used as the subbase. Where bond was lost because of erosion of the subbase, which was the case where the subbase consisted of gravel, the pavement showed distress.

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

3.2 Evaluation of Remaining Life of the Continuously Reinforced Concrete Inlay

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.

3.3 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

3.4 Acknowledgements

The authors would like to thank the organizations that sponsored the project for their support. These include the South African National Roads Agency Ltd (SANRAL), the Gauteng Department of Public Works and Transport (GAUTRANS) and the Cement and Concrete Institute. The permission of the authorities of each of these organizations to publish the findings is acknowledged

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