Improvement of Aggregate Interlock Equation Used in Mechanistic Design Software

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Abstract

The aim of this paper is to present the results of a study, conducted at the University of Pretoria, South Africa, to improve the aggregate interlock equation used in the mechanistic design software, cncPave. It was identified that the previous relationship modelling the mechanism of concrete joints in shear (aggregate interlock) was not accurate, especially for the smaller sized coarse aggregates used in the construction of concrete pavements. A main objective of the study was to investigate existing methods for modelling aggregate interlock shear transfer efficiency and use that as the basis to develop a mechanistic model simulating variations in joint load transfer efficiency with joint opening, load magnitude, subbase characteristics, and concrete aggregate properties. A secondary objective was to investigate the difference in pavement response to static and moving impulse or dynamic loads (equivalent to traffic loads) in terms of deflections across the joint in the pavement. The specific contribution of the study to the improvement of the aggregate interlock equation used in the new mechanistic concrete pavement design method, cncPave, is highlighted in the paper. Some of the conclusions reached after interpretation of experimental results were that the deflection load transfer efficiency was greater during dynamic than static loading, and that larger maximum sized coarse aggregates in the concrete mix (37.5 mm) had lower deflections at the same crack width than smaller sized coarse aggregates (19 mm).

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Introduction

The South African concrete pavement fraternity recognised the need to upgrade the South African Concrete Pavement Design and Construction Manual (Manual M10 1995), to a manual based on mechanistic design principles. It became apparent from overseas research, performance of local pavements and some instrumented sections of a concrete inlay on the National Route 3, that current concrete design methods were inherently conservative and consequently resulted in expensive pavements.

A number of strategy sessions identified that it was necessary to refine and update the existing methods through solid engineering research and to remove unnecessary conservatism that has been documented with existing practice. This approach was fuelled by the fact that concrete pavements are increasingly utilised as overlays/inlays on old flexible pavements where characteristics are determined through linear elastic theory and a mechanistic approach. An integration of flexible and rigid mechanistic design approaches is therefore becoming more and more important. The new method was, however, to be pragmatic and aimed at ordinary pavement design practitioners rather than researchers and academics.

Simultaneously it was also decided to develop a simple, user-friendly software programme as part of the manual. Among the critical input parameters that were identified at the outset of this whole process, was an aggregate interlock load transfer constant, Ca (Strauss et al. 2001). The fact that the format of the aggregate interlock equation itself was still uncertain was emphasized and it was recognised that further research was required to ascertain a more accurate, mechanistic formula.

The aim of this paper is to present the results of a study, conducted at the University of Pretoria to improve the aggregate interlock equation used in the cncPave software. The concept of aggregate interlock is explained, followed by an overview of the research conducted into aggregate interlock in South Africa during the past two decades. A summary of the laboratory programme, the analysis of the data and the application of the model is presented as well as conclusions reached from the study and recommendations for further research.

Aggregate Interlock

Aggregate interlock was first recognised as a beneficial load transfer mechanism in the early 1900s, when the popularity of Portland Cement Concrete as a paving material was beginning to increase. Aggregate interlock is a natural mechanism effective in transferring loads across discontinuities, such as joints and cracks in plain or reinforced concrete pavement systems. Only a shear action is typically assumed to be operative in this mechanism. In contrast, load transfer devices such as dowel bars also involve bending, thus creating an interest to investigate load transfer by aggregate interlock (Ioannides and Korovesis 1990).

Because of its questionable long-term endurance record in the United States of America (USA), it was concluded that "when roughened edges of two slabs are held firmly together the aggregate interlock may be expected to function perfectly and permanently as a load-transfer medium" (Benkelman 1933). Aggregate interlock only is therefore not relied on as a primary load transfer mechanism in jointed concrete pavements in the USA, except perhaps in low volume roads. Abrasion and attrition of the aggregates coupled with temperature variations causing a fluctuation in the size of the opening at the discontinuity can result in a significant decrease in the effectiveness of this mechanism over time (Ioannides and Korovesis 1990).

In an attempt to theoretically model the transfer of shear stress across cracks in concrete, Walraven (1981) showed that the mechanics of aggregate interlock shear transfer is highly complex. In addition to contact between sharp edges of aggregates on joint surfaces, there may be localised crushing of the cement paste and the aggregate, as well as entry of loose materials. The amount of crushing and the bearing area of the surfaces depends on the joint opening, normal restraint developing from the rough surface of the aggregate, the strength of the concrete (both the paste and the aggregate), and the size and distribution of the aggregate particles. Walraven (1981) stated that the modelling of aggregate interlock shear transfer in rigid pavements should take all these factors into account.

During finite element (FE) modelling of aggregate interlock shear transfer in rigid pavement systems, most researchers tend to use discrete linear spring elements. While this may be considered reasonable for an examination of the effect of aggregate interlock shear transfer effectiveness on the global slab response, it does not permit modelling of local response at the joint. Even when the use of linear springs is appropriate, the rational choice of spring stiffness may be difficult, if not impossible, and the appropriate spring value is valid only for one model geometry, set of material properties, and loading. The need for the more realistic FE modelling of aggregate interlock shear transfer was recognised by Davids et al. (1998). They chose the two-phase model developed by Walraven (1981) to model aggregate interlock shear transfer in the FE software EverFE.

A literature review, distinguishing between theoretical modelling, laboratory studies and field investigations has been conducted to provide an overview of past attempts to model this phenomenon and to explain the mechanics of aggregate interlock. The focus of the literature review was to investigate methods used during previous studies, to assist in the design of the experiments, and the compilation of the test programme for the current study.

Aggregate Interlock Research in South Africa

A study was initiated in 1988 to investigate the performance of rigid pavements in South Africa and to develop design techniques based on South African experience (Strauss 1992). The research study involved a field evaluation, laboratory study (Malan et al. 1988) and the development of a modelling technique based on finite element analysis.

During modelling of the pavements under investigation, significant differences in behaviour occurred for loading at the interior and loading at an edge or corner of a slab, emphasising the fact that edge load transfer had a marked influence on the performance of the pavement.

Evaluating the effectiveness of aggregate interlock for different types of mixes, an increase in relative movement (the difference between the deflection in the loaded slab and the simultaneous deflection in the unloaded slab) was obtained with an increase in crack width. For the same crack width, concrete that cracked at 28 days (where some aggregate particles were cracked through) allow ten times more relative vertical movement before aggregate interlock was enacted than concrete cracked at 3 days of concrete age (where most of the cracks went around the aggregate). The same applied for concrete mixes with 25 mm aggregate, which showed 30 times more relative vertical movement than concrete with 50 mm aggregate.

From this study, Strauss (1992) compiled an equation to predict the relative movement (RM) that takes place at an aggregate interlock joint in a concrete pavement, as follows:

$$y(x) = 114000 \ \frac{x^3}{agg^{4,5}} \tag{1}$$

where: *y(x)* = Relative vertical movement (mm);
x = Crack width (mm); and
agg = Nominal size of 20% biggest particles in concrete mix / maximum aggregate size (mm) in concrete mix.

The research carried out by Strauss (1992) evolved into the production of Manual M10 (1995). Most of the design curves used in this manual were developed using linear elastic layered theory together with finite element analyses and were confirmed against design procedures in use at the time. The manual essentially followed a “recipe-type” approach to design and used a series of nomograms.

Eq. 1 was adjusted to reflect real life, before being incorporated as the aggregate interlock model of Manual M10 (1995). However, this equation was still only accurate for concrete constructed with a maximum aggregate size of approximately 26.5 mm; the aggregate used in the concrete constructed for the laboratory studies (Malan et al. 1988). This will be demonstrated in more detail in the paragraph dealing with the laboratory test results below.

During 1998 the South African National Roads Agency Limited and the Cement and Concrete Institute organized a task force and provided funding to upgrade Manual M10 (1995), to a concrete pavement design manual based on mechanistic design principles. A re-evaluation of factors affecting riding quality, structural service life, maintenance and rehabilitation needs re-confirmed the prominent effect of joint performance. Eq. 1 was further adjusted, before being incorporated into the software to reflect that the RM may not be only a function of crack width and aggregate size, but also of the number of load applications and the modulus of elasticity of the concrete, as follows (Strauss et al. 2001):

$$y(x) = f\left(\sqrt{n}, \frac{1}{E_C}, \frac{x^{1.5}}{agg}\right) \tag{2}$$

where: *y(x), x, agg* = As defined for Eq. 1;
n = Number of load applications; and
E_c = Modulus of elasticity of the concrete (MPa).

However, it was identified that the information available to accurately model the mechanism of aggregate interlock, and to establish the life cycle from the “bonded” (closely knit crack with high load transfer) to “failed” (wide crack with little load transfer) state was still insufficient. Further research was therefore needed, which resulted in the study whose results are presented in this paper.

Laboratory Programme

The effectiveness of aggregate interlock load transfer at a joint in a concrete pavement depends on load magnitude, number of load repetitions, slab thickness, joint opening, subbase characteristics, subgrade bearing value, and aggregate angularity.

As the texture of the crack face is a function of the coarse aggregate type, size, and gradation, with the surface texture of the crack directly influencing the aggregate interlock and load transfer capacity of the joint (Buch 1998) it was decided to perform a 2-level, 2-parameter experimental design. For this purpose two types of aggregates and two coarse aggregate sizes were chosen. The aggregate types chosen were Granite, with an estimated concrete elastic modulus of 27 GPa for a 35 MPa concrete and Dolomite with an estimated concrete elastic modulus of 40 GPa for a similar strength concrete (Fulton 2001). These aggregates represented the low end and the high end in terms of strength, respectively, of the spectrum of crushed aggregates used in the construction industry in South Africa. The maximum coarse aggregate sizes chosen were 19 mm and 37.5 mm.

In order to minimise the number of variables tested, it was recognised that some parameters had to be kept constant during the experiments. These were:

- Concrete design strength – 35 MPa (standard strength used in the construction of concrete pavements in South Africa).
- Concrete slab thickness – 230 mm (average slab thickness of jointed concrete pavements in South Africa).
- Sand grading - had to be kept constant to ensure that the volume of coarse aggregate would be the same for different aggregate types to render the same aggregate interlock potential at the crack face for similar maximum sized aggregates. The densities of the two aggregates types used differed from each other, and therefore different masses of aggregate were used in the mix designs to obtain the same volume of coarse aggregate in the 19 mm and 37.5 mm mixes, respectively. The grading of the sand and coarse aggregates complied with the requirements of SABS 1083 (1994).
- Subbase support – the construction of the concrete slabs on a re-usable rubber (approximating a Winkler) subbase (The three primary response parameters in the analysis of a slab-on-grade pavement system are deflection (Δ), bending stress (σ), and subgrade stress (q). When the dense liquid foundation model is adopted, the latter may be eliminated because $q = k\Delta$ (Ioannides and Korovesis 1990)).
- Constant temperature during testing – by testing inside a laboratory facility temperature variation could be controlled eliminating curling of the concrete.
- Angle of the fracture face – by inducing the crack in the slab within 24 hours of casting the concrete it was ensured that the angle of the fracture face would be as vertical as possible.

- Dynamic loading frequency – applying the dynamic loading at a constant frequency of 3 Hz, with the interval between the maximum loading of the dynamic loading actuators, simulating a vehicle crossing the joint at 80 km/h (Colley and Humphrey 1967).

Factors that were controlled during the testing were:

- Crack width; in order to determine the crack width up to which load transfer efficiency (LTE) is affected through aggregate interlock caused by specific sized aggregates.
- The foundation support, which was either a continuous (top rubber layer intact) or a discontinuous (top rubber layer cut through) foundation.

Four concrete slabs were cast, as follows:

- Experiment 1 – 19 mm granite aggregate.
- Experiment 2 – 37.5 mm granite aggregate.
- Experiment 3 – 19 mm dolomite aggregate.
- Experiment 4 – 37.5 mm dolomite aggregate.

Apart from the above mentioned four slabs, a fifth concrete slab with a pre-deformed plastic joint former was also cast to investigate the performance of a different type of joint under both dynamic and static loading.

A standard glass light bulb was used to form bubbles in the plastic, and to create an interlock effect, as the plastic sheet itself, created a smooth surface finish at the crack face. Figure 1 shows schematically what the sheet and bubbles looked like. The sheet was 600 mm wide and 240 mm high to fit exactly into the shuttering for the slab, but with a small edge sticking out at the top.

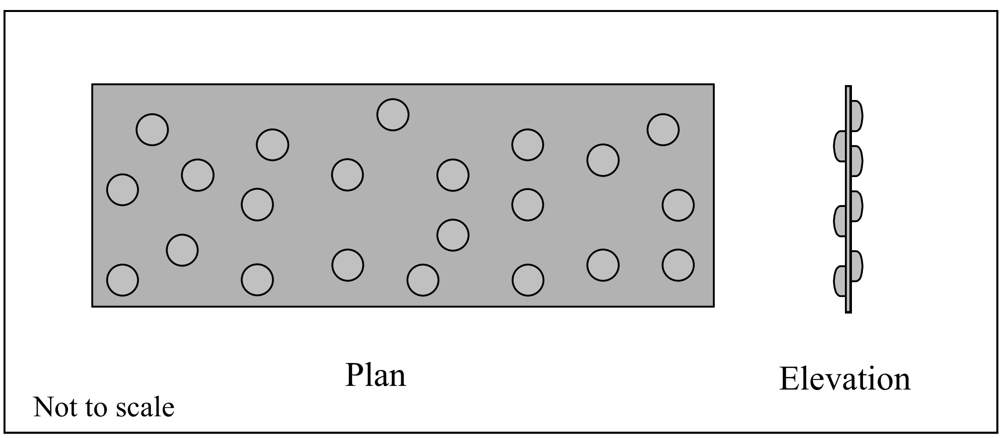

Figure 1: Schematic presentation of plastic joint/crack former

Although the experiment with the plastic joint was not part of the original design, it will be pointed out how the results from this experiment were used to complete the picture of RM versus LTE. Where reference is made in the text, hereafter to a plastic joint, it implies the joint with a pre-deformed plastic sheet as joint former.

In this paper where reference is made to LTE it means deflection LTE, which is the deflection of the unloaded slab over the deflection of the loaded slab, expressed as a percentage. This is different to stress LTE, which is the stress measured in the unloaded slab over the stress measured in the loaded slab, expressed as a percentage.

Test set-up. The concrete beams were cast on approximately 55 mm thick rubber to simulate a dense liquid Winkler foundation and provide a uniform subgrade with continuous support. When tested in a California press to determine the equivalent bearing capacity of the rubber, it was measured as 24.This is equivalent to a selected gravel layer with a resilient modulus of approximately 150 MPa (Theyse et al. 1996) and a k-modulus of 80 MPa/m.

The beam was 1 800 mm long, 600 mm wide, and 230 mm thick. The rubber and shuttering were placed on a timber pack (2 800 mm long, 700 mm wide, and 140 mm thick), where after the concrete beam was cast. The timber pack had to render a sound base for transporting the beam from the position where it was cast to where it was tested.

A crack inducer in the form of an angle iron was placed across the beam at mid-length on the rubber foundation. The crack/joint had to be formed within 24 hours after casting the concrete. For Experiment 1 a 40 mm deep incision was cut into the concrete surface with a grinder (vertically above the angle iron), where after the desired crack was formed. For Experiments 2, 3 and 4 the incision was formed by casting a flat bar into the top of the concrete slab vertically above the angle iron. A schematic layout of the test set-up is given in Figure 2.

More detail on the test set-up and methodology used to obtain load, deflection and temperature data can be obtained from papers already presented on the subject (Hanekom et al. 2001a; Hanekom et al. 2001b; Hanekom et al. 2003). It has also been described in a technical paper (Brink et al. 2003) and a PhD thesis (Brink 2003).

Material tests. The slabs were cast with Granite and Dolomite aggregates obtained from crushing plants situated close to Pretoria in Gauteng, South Africa. To ensure that a 28-day compressive strength of 35 MPa would be achieved, test cubes were made up beforehand, using water/cement ratios of 0.59 and 0.63. The test cubes were crushed after 7 days, and the 28-day strengths were calculated from the assumption that the 7-day compressive strength is approximately two-thirds that of the 28-day compressive strength (Fulton, 1994). The average 7-day compressive strength values obtained for water/cement ratios of 0.59 and 0.63 were 21.5 MPa and 20.5 MPa, respectively, which indicated that the corresponding 28-day compressive strength would be 32.5 MPa and 30.5 MPa. From these results it was determined that a water/cement ratio of 0.56 should be used to obtain a 28-day compressive strength of 35 MPa.

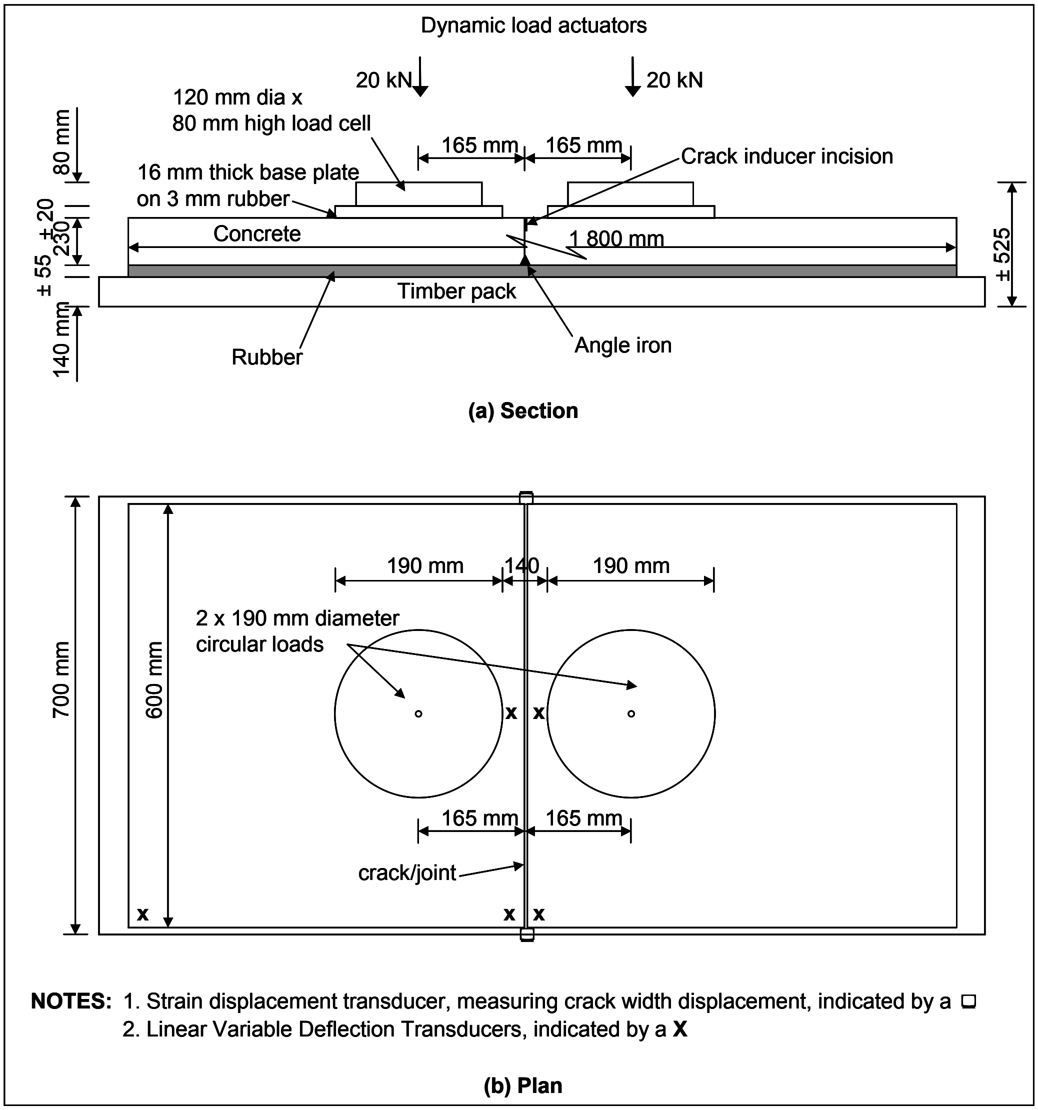

Figure 2: Schematic layout of test set-up

Apart from the slab cast for each experiment, a number of cubes, beams and cylinders were also cast for testing purposes, as summarised in Table 1. Of the 18 cubes cast for compressive strength tests, 9 were water-cured and 9 were air-cured by leaving it in the moulds adjacent to the slab. The actual 28-day compressive strengths obtained varied between 38.7 MPa and 45.0 MPa for water-cured cubes and between 30.0 MPa and 41.7 MPa for the air-cured cubes, respectively.

Table 1: Basic information on cubes, beams and cylinders cast for testing purposes

Test specimen	Dimensions (mm)	Number	Time of test
Compressive strength cubes (SABS 863 1994 / ASTM C39/C39M-01 2001*)	150 x 150 x 150	18	At 7 and 28 days after casting slab, and at end of 2 million load cycles.
Modulus of rupture beams (SABS 864 1994 / ASTM C133-97 1997)	750 x 150 x 150	6	At 28 days after casting slab, and at end of 2 million load cycles.
Shrinkage beams (SABS 1085 1994 / ASTM C426-99 1999)	300 x 100 x 100	4	Measure gauge length L_0 before casting specimen, and L_1 after 7 days in curing bath. Place in drying oven with temperature 50^{o}C, and relative humidity 25%, and measure L_2 at 48 hour intervals thereafter, until difference in length less than 2μm/100 mm.
Modulus of elasticity cylinders (BS1881: Part 121 1993 / ASTM C469-94 1994)	300 x 150 diameter	3	At 28 days after casting.

*NOTE: ASTM test methods give equivalent test results, although the test methods are not necessarily the same.

The volume of 19 mm and 37.5 mm coarse aggregate in both the granite and the dolomite concrete mixes had to be the same, in order to obtain the same aggregate interlock contact areas. In other words, the coarseness of the joint area formed by the 19 mm granite aggregate had to be the same as for the 19 mm dolomite aggregate. The same applied to the 37.5 mm coarse aggregate concrete mixes. To achieve this, the grading of the granite sand and the dolomite sand had to be approximately the same. Figure 3 shows the grading of the sands, within the grading envelope, used in the concrete mixes. The remainder of the material test results has been published as part of a PhD thesis (Brink 2003).

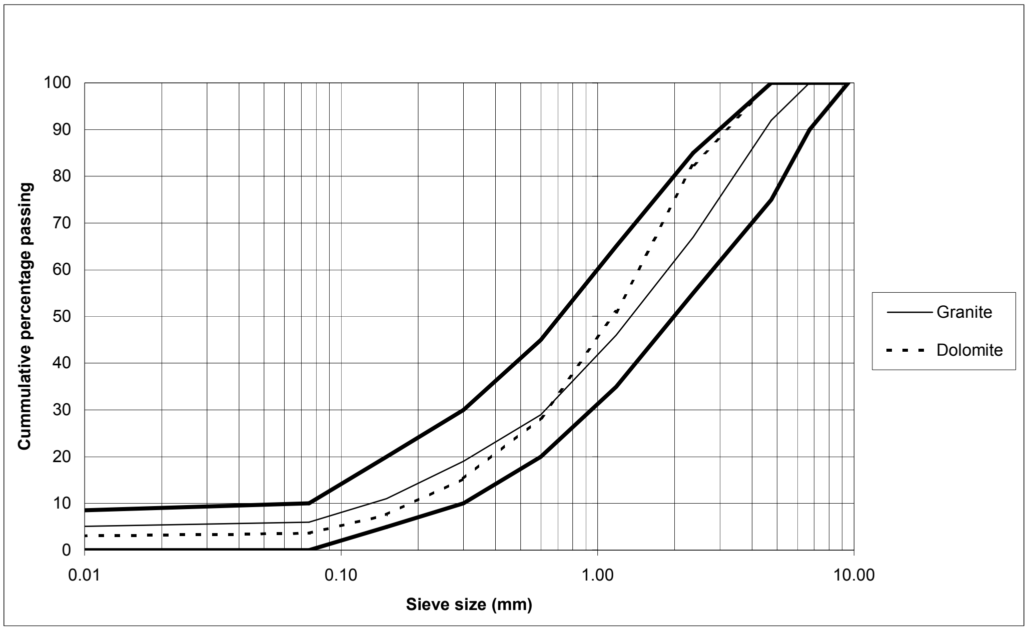

Figure 3: Sand grading

Laboratory Testing Procedure

There were practical aspects of the experimental procedures that needed to be addressed during the first experiment. Initially the intention was to subject all four concrete slabs to at least 2 million dynamic load cycles (Dynamic loading was achieved by using two actuators that simulated the impact of one wheel (20 kN) of a standard 80 kN dual wheel truck axle load, crossing a joint/crack at a speed of 80 km/h through an impulse force.)

It was reasoned that it would be possible to measure deterioration of the crack face through an increase in deflection measurements. The slab cast for Experiment 1 was therefore subjected to 2 million dynamic load applications. After every 0.5 million load applications, static loading tests were carried out, and the data analysed to determine general trends. The equipment was also calibrated at the time of static load testing.

Up to 2 million dynamic load cycles there was no significant deterioration of the load transfer characteristics of the aggregate surface in the crack which indicated that both the cement paste and aggregate particles were so tightly knit together that little vertical sliding, could occur (Benkelman 1933). On the other hand, while applying the dynamic loads, the crack width was also monitored continuously. The relative horizontal opening movement at the crack did increase during application of the 2 million dynamic load applications, yet when the load was removed the actual crack width stayed at 0.1 mm.

This indicated that little abrasion of the aggregates at the joint face took place at the initial crack width, probably due to the fact that the narrow crack restricted vertical shear movement. Therefore no loose particles were dislodged, or got trapped in a different position when the crack opened and closed to act as an agent of abrasion.

Another explanation could be that the shear stress on the crack face at the initial crack width was too low due to lack of support from the rubber subbase. The shear stress at the crack face was therefore calculated using the following equation:

$$\tau = \frac{Agg(RM)}{h} \tag{3}$$

where: τ = Shear stress (MPa);
Agg = Dimensionless joint stiffness per unit length of joint/crack;
RM = Relative vertical movement at joint (mm); and
h = Slab thickness (mm)

The shear stress at the crack face during the initial 2 million dynamic load cycles was calculated as 0.32 MPa for Experiment 1 and as 0.21 MPa for Experiment 2. Compared to the 28-day compressive strength results of 38.7 MPa and 45 MPa for Experiments 1 and 2, respectively and to the elastic modulus results of 21 GPa and 29 GPa, respectively, the shear stress results were low. As mentioned, the equivalent k-value of the rubber subbase, was 80 MPa/m. It was therefore not that the support was not stiff enough, but due to the fact that low RMs were measured during the dynamic loading.

After completion of the dynamic loading cycles the two sections of the slab were pulled apart horizontally in a controlled fashion, to measure responses under static and dynamic loading at different crack widths.

Following on the conclusion already reached from the first two experiments that little or no deterioration of the crack face occurred at the initial crack width during loading, the slabs cast for Experiments 3 and 4 were not subjected to the 2 million dynamic load cycles applied to the first two slabs. The focus was therefore shifted from determining the effect of abrasion under repeated dynamic loading to determining the response of the concrete at different crack widths. The slabs for Experiments 3 and 4 were therefore subjected to only three cycles of 10 minutes of dynamic loading, followed by static loading at the initial crack width. Thereafter, as for the previous two experiments the two sections of the slab were pulled apart and subjected to dynamic and static loading at different crack widths. The slabs were pulled open and tested at different crack widths up to a maximum crack width of 2.5 mm to be on par with similar studies researched during the literature survey (Davids et al. 1998). The slabs were then pushed together again to as close as possible to the initial crack width and pulled open again, at least three times to verify the repeatability of the tests.

These opening and closing cycles to a maximum crack width of 2.5 mm were conducted for all the experiments, except for Experiment 3 that was pulled open to a maximum crack width of 4.0 mm. This was in order to test the assumptions of previous researchers that at crack widths greater than 2.5 mm the deflection measurements reached an upper asymptote. 2.5 mm was also considered the crack width at which the subbase started to play an important role in the LTE of the concrete pavement system (Colley and Humphrey 1967; Davids et al. 1998; Jensen and Hansen 2001).

As mentioned above, a major difference between Experiments 1 and 2, and Experiments 3 and 4 was that the top layer of rubber beneath the concrete was cut through at mid-length directly beneath the joint for the latter, but that all three layers of rubber were left intact for the former.

EverFE (Davids et al. 1998) was used to perform theoretical analyses of the laboratory slab test set-up. A model consisting of the same geometry, material properties, loading and aggregate interlock parameters was tested with EverFE.

Laboratory Testing Results

Deflection. The deflection increased with increasing crack width for both dynamic and static loading as shown in Figure 4. The laboratory results in Figure 4 were less than the theoretical predictions from EverFE. The dynamic loading results were on average 124% that of the static loading results. This was similar to results reported by Bergan and Papagiannakis (1984).

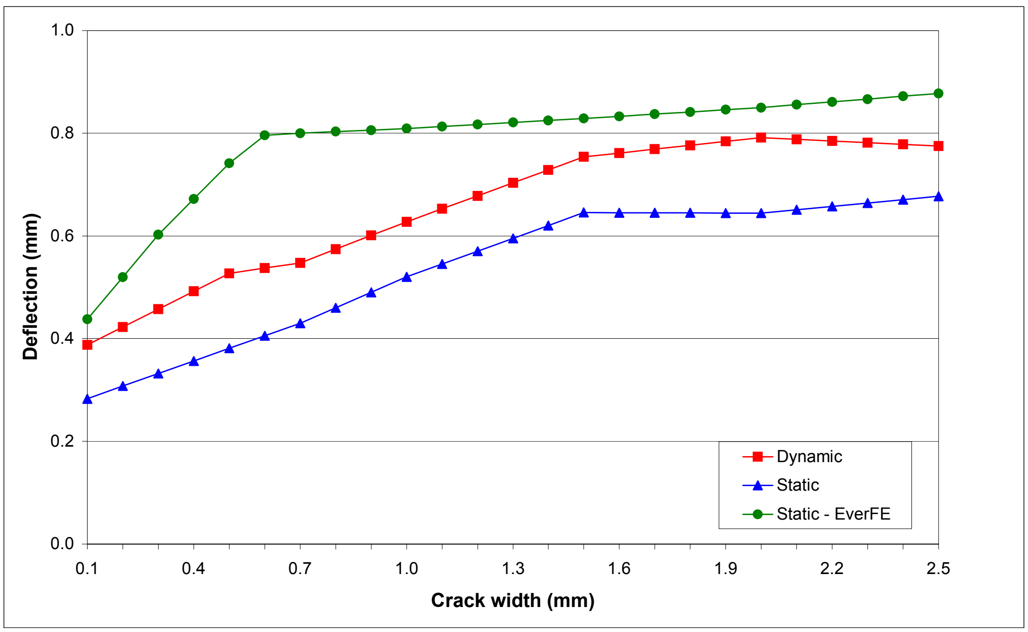

Figure 4: Deflection (of leave slab) versus crack width (19 mm dolomite aggregate)

Load transfer efficiency. The LTE was greater during dynamic than static loading in all instances (see Figure 5). Larger maximum sized aggregates had greater LTE than smaller maximum sized aggregates.

For the same coarse aggregate size concrete mixes, the LTE was larger where there was a continuous rubber support (rubber not cut through) than where there was a crack simulated into the subbase (top rubber layer cut through).

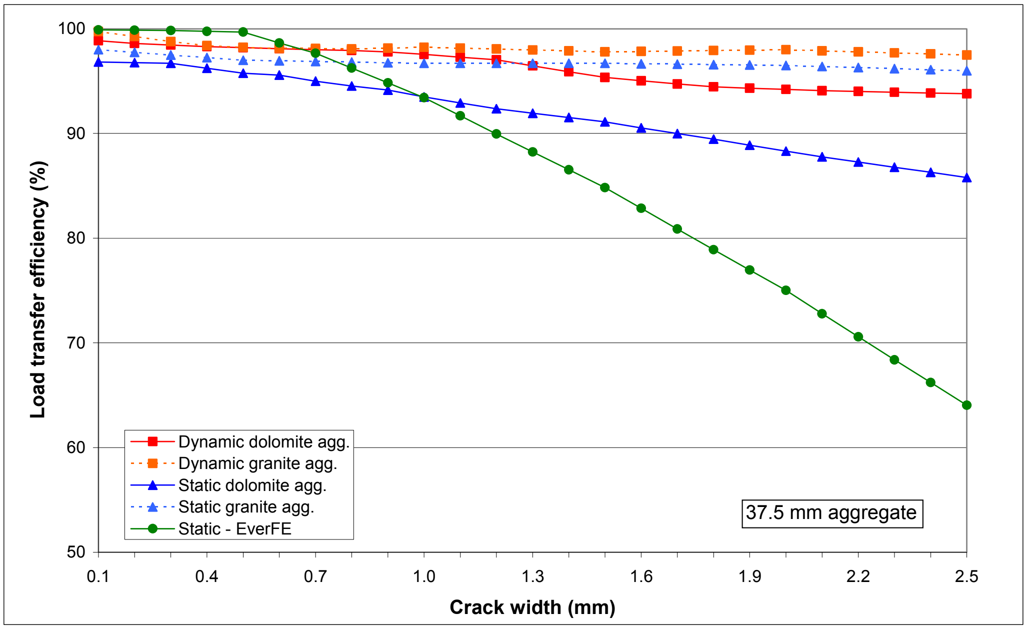

Figure 5: LTE versus crack width – 37.5 mm aggregate

Relative movement. The RMs calculated from the experimental data between the leave and approach sections of the concrete slabs are presented in Figures 6 and 7 for 19 mm and 37.5 mm coarse granite aggregates, respectively. For comparison purposes, the relative vertical movement calculated with EverFE is also plotted on the graphs. As mentioned above, the larger RMs calculated with EverFE, resulting in lower LTEs did not compare well with the experimental results.

The relative vertical movement at the joint for both coarse aggregate sizes considered was also calculated using Eqs. 1 and 2. Although the values calculated for the 37.5 mm coarse aggregate was relatively close to the experimental results, there was a large difference between the values calculated for the 19 mm coarse aggregate and the experimental results, especially for Eq. 1 (Figure 6). The magnitude of the difference in results is more obvious when comparing the scale of the vertical axes of Figures 6 and 7. The former has a maximum value of 3.5 mm, versus the 0.35 mm of the latter.

From Figures 6 and 7 it is obvious that Eq. 1 was inaccurate and needed revision and that Eq. 2 originally incorporated in the cncRisk software was already an improvement. However, the shape of the curve generated using Eq. 2 was still exponential, implying that the larger the crack width the more inaccurate the results would be.

This confirmed that the research conducted during this study was indeed necessary.

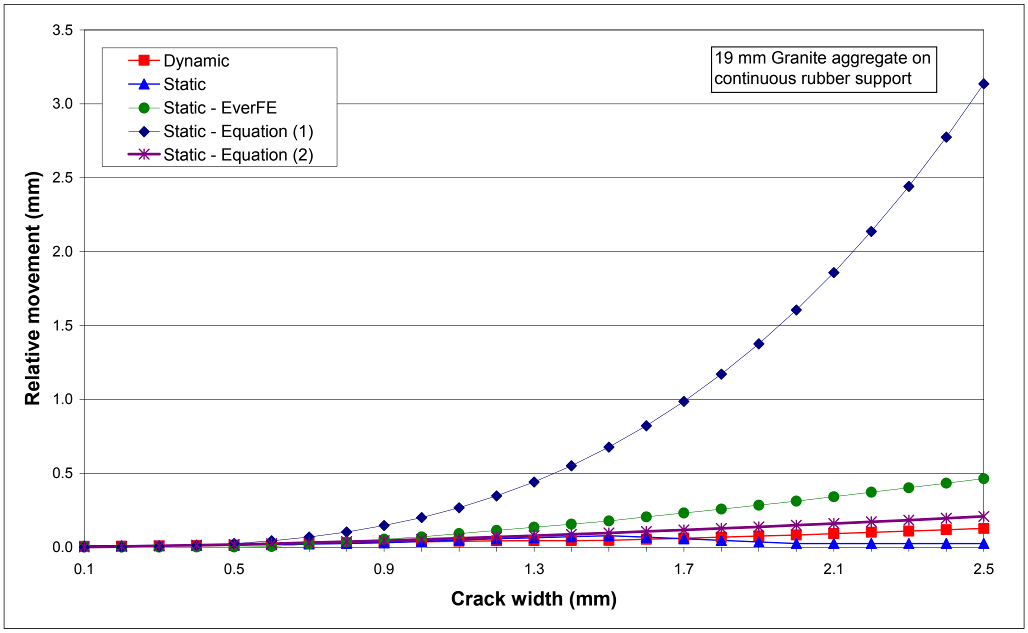

Figure 6: RM versus crack width (19 mm granite aggregate)

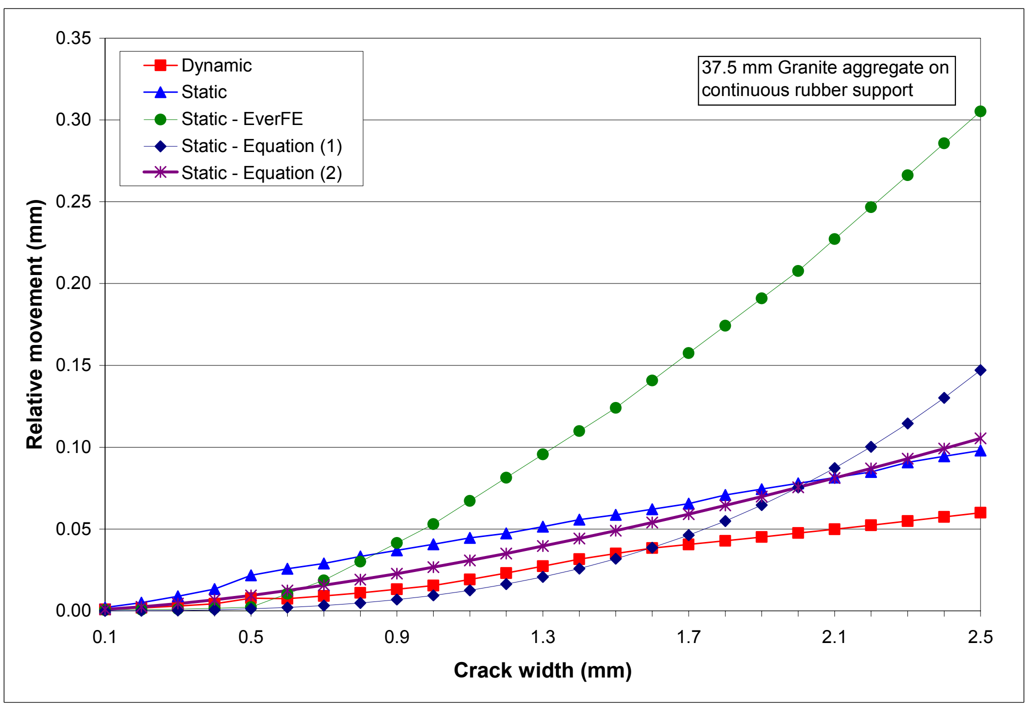

Figure 7: RM versus crack width (37.5 mm granite aggregate)

Improvement of Aggregate Interlock Equation

Laboratory data. The RM data calculated from the deflections was used to develop a single equation to replace Eq. 2 in the source code of the software package. A Weibull probability density function was generated, as follows:

$$y(x) = 0{,}118\left(1 - e^{-\left(\left(v + \frac{11{,}413}{agg}\right)x\right)^{1{,}881}}\right) \tag{4}$$

where: $y(x)$ = Relative vertical movement at joint/crack (mm);
v = 0.136 for static loading (speed = 0 km/h);
= 0.035 for dynamic loading (speed = 80 km/h);
x = Crack/joint width (mm); and
agg = Nominal size of 20% biggest particles in concrete mix (mm).

So-called "before" analyses were conducted with the cncRisk version of the software, containing Eq. 2. For the input on the Main Control Panel of the software, typical jointed concrete pavement parameters were used (see Figure 8).

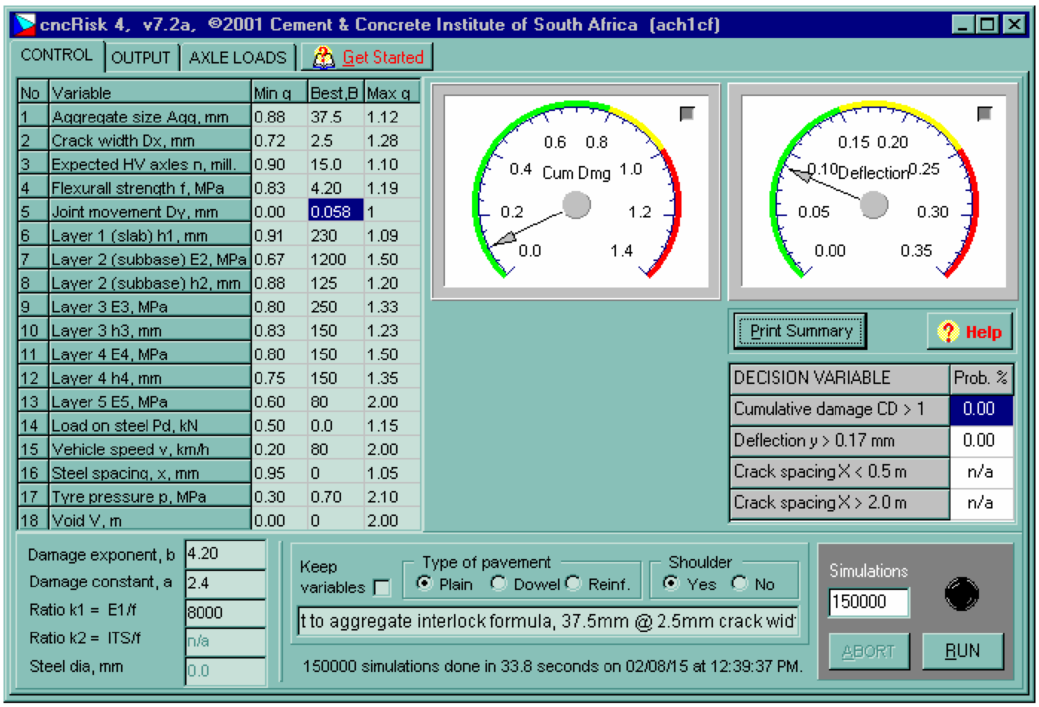

Figure 8: Input detail for Main Control Panel

The variation in the theoretical expected life of the pavement with 19 mm and 37.5 mm coarse aggregate sizes, at specific crack widths was determined. Actual relative movements measured at specific crack widths during the laboratory studies were used.

Eq. 4 was then included in the source code, and the same sets of input data re-used to conduct "after" analyses with the revised version of the software. The results of these "before" and "after" analyses are summarized in Table 2.

Table 2: Comparison between "before" and "after" cncPave analyses

Aggregate size (mm)	Crack width (mm)	Relative movement (mm)	Pavement Life ($N \times 10^6$)	
			Before	After
19	1.0	0.040	46.6	52.6
	2.0	0.092	50.7	48.7
	2.5	0.100	55.1	48.0
37.5	1.0	0.015	54.0	58.2
	2.0	0.042	47.8	52.8
	2.5	0.058	46.8	51.3

For the smaller 19 mm sized coarse aggregate, the expected life calculated through the software increased with increasing crack width, and relative movement during the "before" analyses. This is contrary to what physically happens in a concrete pavement where the life decreases with an increase in crack width and relative movement. The results for the larger 37.5 mm coarse aggregate were more realistic, decreasing with an increase in crack width and relative movement. The pavement life calculated after the inclusion of Eq. 4 decreased with increasing crack width and relative movement, for both coarse aggregate sizes. The pavement life was also higher for the 37.5 mm coarse aggregate concrete than for the 19 mm coarse aggregate concrete, which followed intuition.

The results obtained were to illustrate that the inclusion of Eq. 4 improved the aggregate interlock model in the software and that more intuitively correct results were obtained. However these results should not be taken as absolute values as Eq. 4 was originally tested in cncRisk, which has been improved since and has also been renamed to cncPave.

Field data. The historical data of three in-service concrete pavements in South Africa (with un-doweled aggregate interlock joints) were used to validate the laboratory data. The three concrete pavements were:

- Road Section 1 - concrete overlay (on asphalt) with concrete shoulders, situated in a wet climatic region, in a good condition;
- Road Section 2 - concrete pavement with asphalt shoulders, situated in a wet to moderate climatic region, in a fair condition; and
- Road Section 3 - concrete pavement with concrete shoulders, situated in a moderate climatic region, in a good condition.

During the validation the existing pavement design, field investigations conducted, and a structural evaluation with remaining life predictions for each of the in-service pavements were taken into account. Falling Weight Deflectometer (FWD) data (normalised for temperature gradients) of the road sections were used to calculate the LTE and RM at the concrete pavement joints. A summary of the statistical parameters of the LTE and RM data for the three road sections is given in Table 3.

Table 3: Summary statistics of LTE and RM data for Road Sections 1, 2, and 3

Statistical parameter	**Road Section 1**		**Road Section 2**		**Road Section 3**	
	RM	**LTE**	**RM**	**LTE**	**RM**	**LTE**
Average	0.013	87.3	0.126	57.4	0.146	42.2
Standard deviation	0.012	11.5	0.120	25.5	0.109	20.6
Minimum	0.000	57.4	0.002	9.4	0.000	4.9
Maximum	0.049	100.0	0.586	99.0	0.495	100.0

At the time of the investigation, the ages of Road Sections 1 to 3 were 14, 23, and 13 years, respectively. Road Sections 1 and 3 were constructed at approximately the same time, and carried the same amount of traffic, and it could have been expected that their remaining lives would be the same. However, the concrete pavement on Road Section 1 was an overlay on an existing asphalt pavement. The subbase beneath the concrete of Road Section 1 was therefore more elastic than the cement stabilised subbase beneath the concrete pavement of Road Section 3. The result of this was lower deflections and RMs for the former than for the latter at the joints, but higher mid-slab deflections. The remaining life of Road Section 1 was 14 years and of Road Section 3, 20 years. Other factors that contributed to the lower remaining life of Road Section 1 were poorer subgrade conditions, and a wetter climate subjecting the pavement to not only mechanical weathering due to traffic loading but also chemical weathering processes.

In terms of design age and RM, it could be expected that Road Section 2 would still have a structural capacity of 8 to 10 years (assuming a 30 year design life (TRH4 1985)). However, as mentioned this road section had an asphalt shoulder, which were already cracked and showing signs of ageing. Furthermore, there was a no-fines subsoil drain between the concrete pavement and the asphalt shoulder, which failed and also allowed water to enter the pavement. This contributed to a reduction in the remaining life, which was calculated as only 6 years.

The calculated RM versus LTE data together with an exponential curve fitted through the points for each road section is shown in Figure 9. The curves fitted through the data points were all of the format $y(x) = 100e^{-Fx}$, with $RM(x) = 0$ at $LTE(y) = 100\%$. The intercept of all three curves was 100. The factor, F, was assumed to be a function of the subbase support, the radius of relative stiffness, as well as the subgrade k-modulus, which still had to be determined.

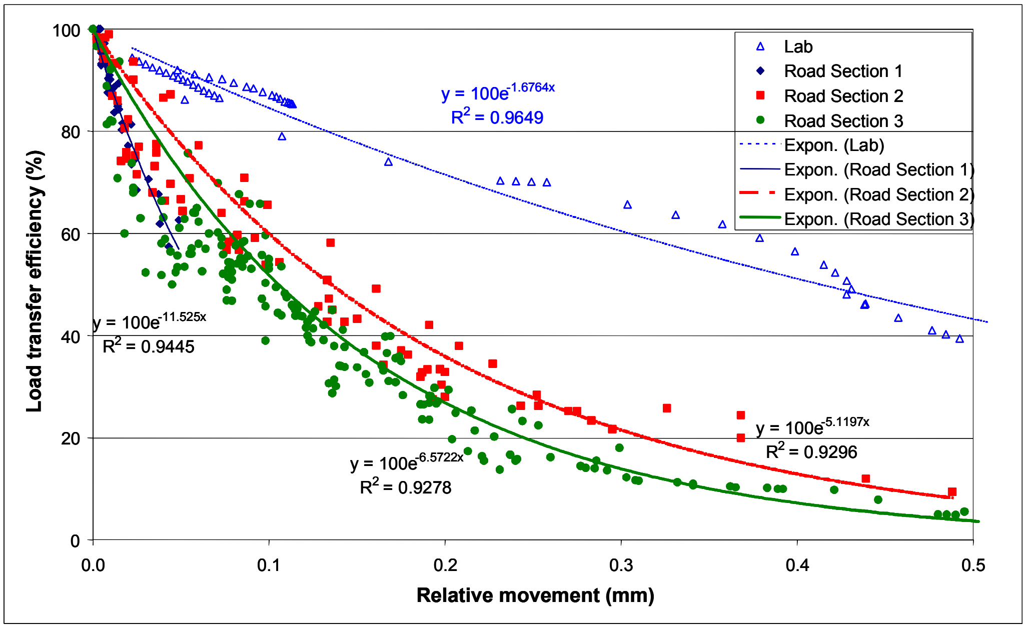

Figure 9: RM versus LTE for Road Sections 1, 2, and 3

Contrary to the maximum RM of approximately 0.12 mm measured in the laboratory for the aggregate interlock experiments, values of up to 0.5 mm were measured in the field. However, it has been mentioned that the results from testing across the plastic joint were used in the validation process. The 40 kN static loading data for the 19 mm and 37.5 mm coarse aggregates, combined with the results obtained from testing across the plastic joint, covered the same range of RM measurements as the field data. The combined results together with the exponential curve fitted through the data points are also plotted on Figure 9.

Of the three sections, Road Section 3 supposedly has the largest RM, but according to the exponential curve fitted to the data, it approximates a lower LTE at a RM of 0.5 mm than Road Section 2.

Road Section 2 with the lowest RM of the three sections considered showed the highest LTE. The amount of data available for Road Section 1 was small in comparison with Road Sections 2 and 3. Most of the LTE results for Road Section 1 were between 60% and 100%, with the maximum RM only 0.05 mm.

When analysing the results plotted on Figure 9, the sensitivity of the model developed in this study can be summarised as follows:

- The larger the crack width, the greater the RM;
- The greater the vehicle speed, the smaller the RM;
- The larger the aggregate size, the smaller the RM;
- The higher the elastic modulus of the concrete the smaller the RM for the same LTE; and
- The higher the elastic modulus of the subbase, the smaller the RM, also for the same LTE.

Various combinations of the subgrade modulus (*k*), radius of relative stiffness (*l*) and the subbase stiffness ($E_{subbase}$) were used in order to calculate a "shift" factor between the laboratory and the field data. For the equation $LTE = e^{F*RM}$, the combination that gave the closest results for the shift factor (*F*) was:

$$F = l/(k*E_{subbase}) \tag{5}$$

The *F* of the exponential curve fitted to the laboratory data in effect already incorporated these three parameters as an inherent characteristic. It was therefore logical that these three parameters would also contribute to the shift in data. The shift factor by itself has the units mm^2/MPa^2, which is a surface area over a surface stress. When, however the exponential curve fitted to the laboratory data is divided by the shift factor, the units are cancelled out, making it dimensionless.

Another important aspect that has to be emphasised here and which the range achieved in the laboratory RM shows, is that the RM over which aggregate interlock plays the primary role in load transfer, varies from approximately 0 mm to 0.12 mm. At larger RMs the subbase influenced the results to a great extent, which is clearly shown by the RM of the smooth plastic joint. This was also one of the main reasons for including the stiffness modulus of the subbase during determination of the F-factor.

Where the RM in the field is larger than 0.12 mm, two main assumptions can be made from this comparison between the laboratory and the field data, namely:

- The aggregate interlock capacity of the crack face, itself is still intact, but the crack width is larger than 2.5 mm, or
- The crack face has been eroded due to traffic loading to such an extent, that the crack face has become smooth, and has little aggregate interlock capacity.

Conclusions

Some of the conclusions reached after interpretation of experimental results were as follows:

- An increase in crack width caused an increase in deflection, a decrease in LTE, and an increase in RM as would intuitively be expected.
- The larger 37.5 mm aggregate had lower deflections than the smaller 19 mm aggregate at the same crack widths during dynamic and static loading.
- Beyond a crack width of 2.5 mm the data for the 19 mm coarse aggregate tended to remain constant, and it was therefore not considered necessary to test at crack widths greater than 2.5 mm. It was specifically stated in previous research studies (Davids et al. 1998; Jensen and Hansen 2001) that at crack widths greater than 2.5 mm the stiffness of the subbase starts to play a role in levelling out the measured response of the slabs. However, this study has shown that the smoother the texture of the crack face, the sooner the system would rely on the support of the subbase to transfer stresses and strains from one slab to another. This study has indicated that three such transition zones exist, namely: 1.5 mm for the smooth plastic joint, 2.5 mm for the 19 mm aggregate interlock joint, and between 3.5 mm and 4.0 mm for the 37.5 mm aggregate interlock joint.

- The LTE was greater during dynamic than static loading in all instances. Larger sized coarse aggregates had greater LTE than smaller sized coarse aggregates.
- For the same coarse aggregate size concrete mixes, the LTE was larger where there was a continuous subbase support (rubber not cut through) than where there was a crack simulated into the subbase (top rubber layer cut through).

Recommendations

The improved aggregate interlock equation developed during this study has already been incorporated in the cncPave software. The software can be used with confidence.

An aspect that has not been specifically addressed during the present study was the effect of abrasion or aggregate wear out within the aggregate interlock joint, as little abrasion took place during application of the 2 million dynamic load cycles at the initial crack width. As mentioned, this was partly due to the small relative vertical movement that could take place in the still "locked-up" state of the crack. Abrasion within the aggregate interlock joint has however been observed at the concrete pavement trial sections constructed next to the National Route 3 at Hilton in Kwazulu-Natal, where accelerated pavement testing was conducted using a Heavy Vehicle Simulator (HVS) testing machine. The data obtained from Hilton has been used to calibrate the aggregate interlock equation that has been developed during this study and to further improve cncPave.

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NOTE:
Where reference is made to Hanekom, A.C., it is the same entity as Brink, A.C. The former is the married name and the latter the maiden name of the main author.