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Improvement of aggregate interlock equation for new mechanistic concrete pavement design method

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In her thesis 'Modelling aggregate interlock load transfer at concrete pavement joints' she addressed an important aspect of the input required to upgrade the South African *Concrete pavement design and construction manual* to a design method based on mechanistic design principles. Conclusions reached from the study were inter alia that there was no significant deterioration of the crack face during application of up to two million dynamic loads. This indicated that fatigue of the aggregates at the joint face does not play a role, but it could also be attributed to the high-quality crushed stone used in South Africa. It was further established that the potential of a jointed concrete pavement to transfer loads across a joint/crack should rather be evaluated in terms of relative movement than in terms of deflection load transfer efficiency, which has been the method used in practice up till now. A specific relationship determined from laboratory results was used to refine the aggregate interlock load transfer model in the software developed for the new mechanistic design manual. This relationship was also used to interpret data from field investigations of in-service concrete pavements in South Africa.

This paper presents aspects of an investigation into methods for modelling aggregate interlock shear transfer in jointed concrete pavements. The aim was to improve the aggregate interlock load transfer equation used in the software developed for the new South African mechanistic concrete pavement design method. The paper presents the evaluation of the results of the investigation into aggregate interlock load transfer efficiency across joints in concrete slabs subjected to simulated 20 kN dynamic wheel loads and 40 kN control loads. One of the main conclusions reached from the study was that there was no significant deterioration of the crack face during dynamic loading, which indicated that fatigue of the aggregate interlock at the joint face did not play a role. This could be attributed to the high quality of the crushed stone used in South Africa. The equation developed from the laboratory results was used to refine the aggregate interlock load transfer model in the software developed for the new mechanistic design method.

INTRODUCTION

During the upgrading of the South African *Concrete pavement design and construction* manual M10 (Manual M10 1995) to a concrete pavement design method based on mechanistic design principles, a re-evaluation of factors affecting riding quality, structural service life, maintenance and rehabilitation needs reconfirmed the prominent effect of joint performance. It was identified that the current relationship modelling the mechanism of concrete joints in shear (aggregate interlock) was not accurate enough, especially for the smaller-sized coarse aggregates used in the construction of concrete pavements. Further research was therefore needed, which resulted in a study whose results are presented here.

A main objective of the study was to investigate the applicability of existing methods for modelling aggregate interlock shear transfer efficiency in order to determine a fundamental 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 differential deflections across the joint in the pavement.

The aim of the paper is to present an aspect of an overall study, namely the evaluation of the results of the investigation into deflection load transfer efficiency due to aggregate interlock across a joint in 35 MPa concrete slabs. Coarse aggregate sizes of 19 mm and 37,5 mm were used to cast the experimental slabs. A total of five slabs were cast. The first four (experiments 1 to 4) were primarily to investigate the aggregate interlock potential of the aggregates as follows:

- Experiment 1: 19 mm granite aggregate on a continuous rubber subbase
- Experiment 2: 37,5 mm granite aggregate on a continuous rubber subbase
- Experiment 3: 19 mm dolomite aggregate on a discontinuous rubber subbase
- Experiment 4: 37,5 mm dolomite aggregate on a discontinuous rubber subbase

The fifth slab (experiment 5) was used to quantify the effect of subbase support.

The data obtained were compared with results obtained from modelling with theoretical three-dimensional finite element (3D FE) software (Davids *et al* 1998).

AGGREGATE INTERLOCK

Ever since the early 1900s researchers have been questioning the mechanism of aggregate interlock in concrete pavements. Various experimental studies on aggregate interlock shear transfer in concrete pavements demonstrated that joint shear transfer effectiveness and endurance depended on factors such as joint width, slab thickness, load magnitude, foundation type, subgrade modulus, and aggregate shape (Colley & Humphrey 1967).

Research by Walraven (1981) into the more general problem of shear transfer across discrete cracks in concrete 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 depend on the joint opening, normal restraint of the joint, the strength of the concrete (both cement paste and the aggregate), and the size and distribution of the aggregate

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particles. Modelling aggregate interlock shear transfer in rigid pavements should take all these factors into account. Cumulative damage to the joint due to cyclic loading reduces the ability of the joint to transfer shear (Walraven 1994).

In undowelled pavements, aggregate interlock is the main shear load transfer mechanism at transverse joints, and is quantified in terms of deflection load transfer efficiency (LTE_Δ), which is defined as the ratio of the deflection of the unloaded slab (Δ_U) to the deflection of the loaded slab (Δ_L) as follows:

$$LTE_\Delta = \Delta_U/\Delta_L \qquad (1)$$

Load transfer efficiency at the joint can vary with concrete pavement temperature, age, moisture content, construction quality, magnitude and repetition of load and type of joint (Hammons & Ioannides 1996). As mentioned, the main objective of the study was to model the aggregate interlock shear transfer efficiency at the different crack widths that would normally be induced by these factors.

Davids *et al* (1998) incorporated an aggregate interlock model developed by Walraven (1981) in the development of the three-dimensional computer program EverFE. They used a 16-noded iso-parametric joint element to incorporate the crack constitutive relations in the finite element models, permitting the effects of joint opening and concrete properties to be captured.

Davids *et al* (1998) admitted that field validation of their mostly theoretical models were still required. Further research was therefore required, especially into the aggregate types used in South Africa, before applying EverFE with confidence.

LABORATORY STUDIES

Test set-up and methodology

The test set-up and methodology used to obtain load, deflection and temperature data have already been described in detail in two papers (Hanekom *et al* 2001a; Hanekom *et al* 2001b).

Although the methodology is described in the above references, it is important to highlight why the concrete slabs were tested under dynamic as well as static loading. The falling weight deflectometer (FWD) measuring instrument is commonly used to determine deflection load transfer efficiency at a joint in a concrete pavement. With the FWD, a static impulse load is applied by dropping a load onto the pavement on the one side of a crack/joint, and measuring the deflection on both sides. The deflection load transfer efficiency is then calculated using equation 1. In practice, however, the loads applied to a pavement are not 'one-sided', but moving dynamic loads are transferred from one slab to the next under vehicle traffic. The response of the slabs therefore had to be captured under dynamic loading (with two actuators) as well as under static loading (with one actuator, similar to FWD) to reflect real-life conditions and to compare the results.

The slabs were cast on approximately 55 mm thick rubber (three layers) to simulate a dense liquid (Winkler) foundation. For experiments 1 and 2, the rubber beneath the concrete provided a continuous support, whereas for experiments 3 and 4 the top layer of rubber was cut through right beneath the crack formed at mid-length in the concrete to simulate a crack propagating into the subbase.

Material properties

In order to quantify the role of the type of aggregate used when evaluating the aggregate interlock potential of the crack face, granite and dolomite aggregates were chosen as they represented the lowest and highest elastic moduli, respectively, of the crushed aggregates used in the construction of concrete in South Africa. Granite represented the low end of the spectrum, with an estimated concrete elastic modulus of 27 GPa for a 35 MPa concrete, and dolomite the high end, with a concrete elastic modulus of 40 GPa for a similar strength concrete (Fulton 2001). The slabs were cast using both 19,0 mm and 37,5 mm granite and dolomite aggregate. Apart from the slabs, a number of cubes, beams and cylinders were also cast for each experiment, as summarised in table 1. As the slabs were air-cured, the number of cubes cast was duplicated in order to compare the compressive strength of the air-cured cubes left adjacent to the slab, with that of the water-cured cubes. The test results are summarised in table 2.

The concrete was designed to achieve a 28-day compressive strength of 35 MPa. This was generally obtained, except for the air-cured cubes of experiment 1 where only 30 MPa was measured. The modulus of rupture test results were more or less 10 % of the compressive strength results, and the amount of shrinkage measured was negligible. The modulus of elasticity for the 19 mm granite aggregate mix was lower than the elastic modulus of the source aggregate, whereas the other modulus of elasticity results were all higher than that of the source aggregate.

Analysis of results

Deflection

Crack width and subbase support were found to be the primary factors controlling slab deflection. This is in agreement with previously published work (Colley & Humphrey 1967). A typical example of the increase in deflection with increasing crack width is shown in figure 1. For purposes of comparison the EverFE theoretical analysis results are also plotted on the

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 °C, and relative humidity 25 %, and measure L_2 at 48-hour intervals thereafter, until difference in length less than 2 µm/100 mm
Modulus of elasticity cylinders (BS1881: Part 121: 1993 / ASTM C469-94, 1994)	300 x 150 diameter	3	At 28 days after casting

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

Table 2 Laboratory test results

Experiment number	Curing method	Compressive strength (MPa) at			Modulus of rupture (MPa) at		Shrinkage (%)	Modulus of elasticity (GPa)
		7 days	28 days	Time of test*	28 days	Time of test*		
1	Water	24,5	38,7	50,0	4,75	4,90	0,019	21,0
	Air	20,0	30,0	36,7				
2	Water	27,2	45,0	57,5	4,40	5,00	0,018	29,0
	Air	29,0	41,7	50,0				
3	Water	27,7	41,5	55,0	5,00	5,10	0,016	41,2
	Air	27,7	38,5	48,2				
4	Water	29,8	43,5	49,3	4,80	5,03	0,035	48,0
	Air	27,5	39,3	45,0				

*Notes:

1 The **time of test** for experiments number 1 and 2 was after the application of 2 million dynamic load cycles, at 66 days and 138 days after casting, respectively.

2 For experiments 3 and 4 it was at the commencement of the testing at different crack widths, at 133 days and 120 days after casting, respectively.

3 The slab for experiment 5 was cast using the same concrete mix design as for experiment 1. Only the 7-day cube compressive strength was determined and a forecast made of the 28-day results. This was only for control purposes, as the properties of the concrete for this specific mix design have already been determined through the results listed above.

graph. On the continuous rubber subbase, the laboratory deflection results were approximately three times higher than the numerical deflection results at a crack width of 2,5 mm. On the discontinuous rubber subbase, on the other hand, the laboratory results were approximately 80% of the numerical results at the same crack width.

The main reason for the difference between the laboratory results and the theoretical results can be attributed to the fact that EverFE uses a linear elastic model for the pavement layers, whereas the rubber used in the experiments simulated a Winkler foundation. EverFE can model any number of linearly elastic subbase/soil layers to any depth beneath the slab. Below the bottom-most layer EverFE uses a Winkler foundation, recognising that foundation stresses at this depth are low and will have little effect on the slab response (Davids *et al* 1998). Although the elastic solid foundation is often considered a more realistic soil representation, it has not been used extensively in concrete pavement analysis and design. This is probably because elastic solid is a continuum model, and sometimes fails to simulate the behaviour of real soils, which are particulate media, especially under conditions of edge and corner loading (Khazanovich & Ioannides 1993).

On the other hand, the difference between the deflection results measured on the continuous rubber and those measured on the discontinuous rubber could be attributed to the fact that the continuous rubber transferred more of the load through the subbase. The discontinuous rubber inhibited the transfer of load through the subbase and the deflection of the loaded slab was therefore less.

Load transfer efficiency

In a 4,5 m long slab a 10 °C increase in temperature can cause a 0,5 mm elongation of the slab. Simultaneously, this increase in temperature can cause curling (convex shape with centre of slab lifting up) in the slab of up to 1,33 mm (Huang 1993). Crack widths of up to 2 mm have been measured in a jointed concrete pavement with 6 m joint spacing (Wattar *et al* 1999). An increase in slab length, combined with curling of the slab, would therefore decrease the crack width (when two adjacent slabs increase in length) and relative (vertical) movement at the crack and thereby increase the load transfer efficiency. The opposite is also true. A decrease in slab length, combined with warping (concave shape with edges lifting up) of the slab, would increase the crack width and relative movement at the crack and thereby decrease the load transfer efficiency.

Equation 1 was used to calculate the load transfer efficiencies from the deflections measured in the laboratory. For demonstration purposes, a comparison between the dynamic and static loading values calculated for experiments 3 and 4 are plotted in figure 3.

The slabs cast with the larger 37,5 mm coarse aggregate showed greater deflection load transfer efficiency than the slabs cast with the smaller 19 mm coarse aggregate. This held true for the slabs cast with both the granite and the dolomite aggregates. The results predicted by EverFE as a function of crack width once again differed from the laboratory results. EverFE predicted load transfer efficiencies close to 100 % up to a crack width of 0,5 mm, whereafter the theoretical results decreased to values far less than what was measured in the laboratory.

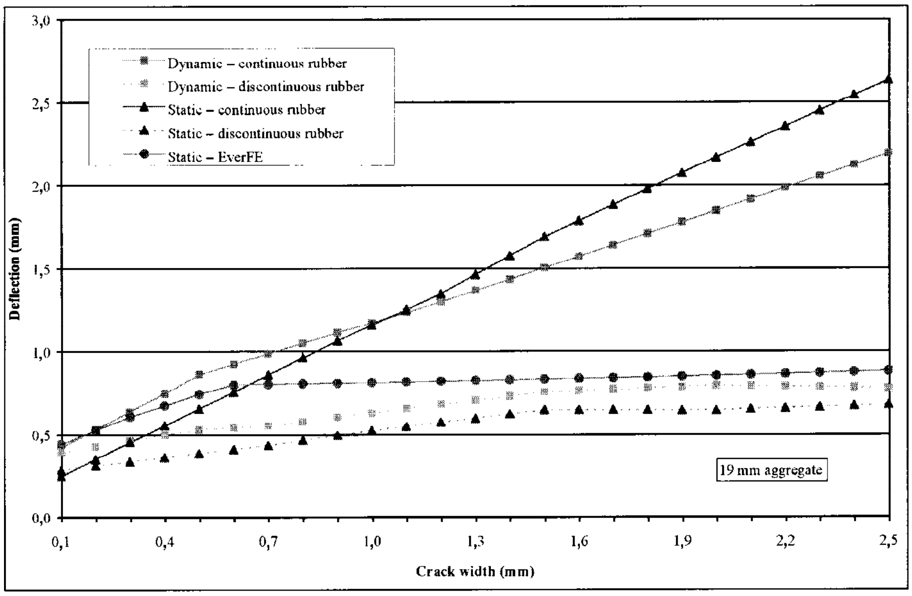


Figure 1 Deflection versus crack width (19 mm aggregate)

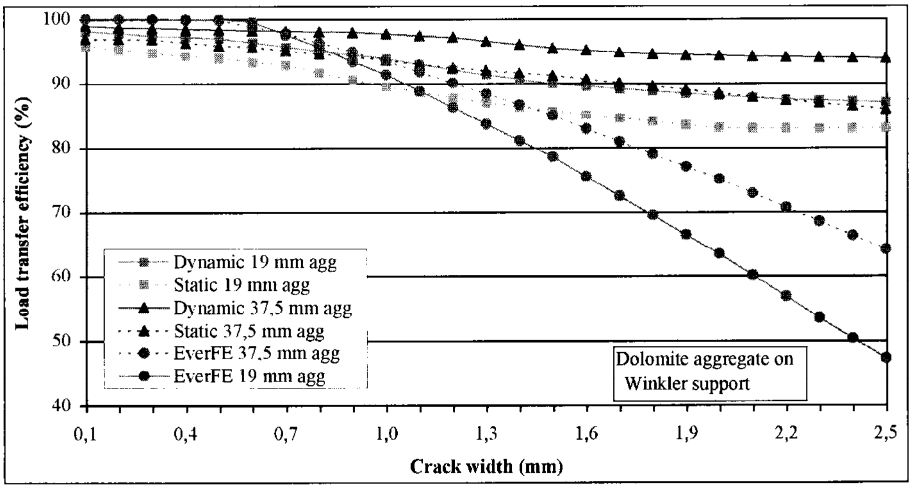


Figure 2 Deflection load transfer efficiency versus crack width (19 mm and 37,5 mm dolomite aggregate)

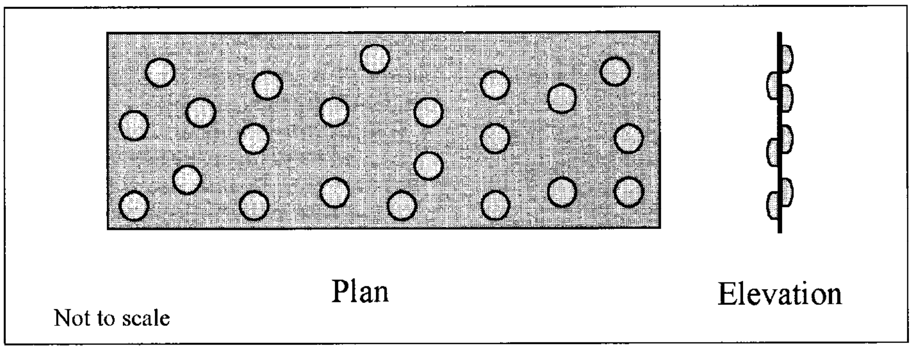


Figure 3 Schematic presentation of plastic joint/crack former

Relative movement

The deflection results obtained on the four slabs were further compared in terms of relative movement. Although the two slabs were on different subbase support systems, the relative movements calculated between the leave and approach slab deflections for the 19 mm granite were similar to those calculated between the leave and approach slab deflections for the 19 mm dolomite aggregate. The same applied to the relative movements calculated from the results of the tests using the 37,5 mm granite and dolomite aggregate concretes.

The assumption was therefore made that the relative movement results for the same coarse aggregate size concrete mixes could be combined. This was quantified by means of the fifth experiment.

The slab cast for the fifth experiment had a pre-deformed plastic sheet as the joint former and not the aggregate interlock crack faces induced for the other four experiments (see figure 3 for a schematic layout of the plastic sheet). Furthermore, the fifth slab was subjected to the same set of tests as the other four experiments, but on both subbases. This was achieved by firstly conducting the full set of tests on a discontinuous rubber foundation (top layer of rubber cut through). The two parts of the slab were then tightly held together, the slab lifted off the rubber and the top layer of rubber replaced with a continuous piece of rubber. The full set of tests was then repeated on the continuous rubber foundation.

When comparing the relative movements calculated from the deflection results, the results obtained under dynamic loading for both subbases tested were remarkably close, as shown in figure 4.

The smooth pre-deformed plastic joint surface had little interlock capacity and it could therefore be expected that the relative movements under static loading would not be similar. As the continuous rubber subbase had a greater load transfer efficiency under static loading than the discontinuous rubber subbase, it had the ability to reduce relative movement between the two sections of the slab, thereby yielding smaller relative movements for the continuous rubber subbase than for the discontinuous rubber subbase.

An important point that needs to be stressed here is that, owing to the smooth surface texture of the pre-deformed plastic joint, the static loading results were different; however, during analysis of the results obtained for experiments 1 to 4, the static loading results for the same maximum aggregate size concrete mixes were similar, and could therefore be combined. This can be attributed to the coarseness of the aggregate interlock crack faces. This factor was quantified by conducting volumetric surface texture tests on all relevant samples, as described below.

A conclusion that could be drawn from the fifth experiment was therefore that it is more accurate to quantify the efficiency of an aggregate interlock joint to transfer load in terms of relative movement than to quantify it in terms of deflection load transfer efficiency. The main reason is that the former method isolates the concrete itself, whereas the latter incorporates the pavement system as a whole (Strauss 2001). However, this approach is not logical as there are three components involved during load transfer at a joint/crack, namely the portion carried by the slab, the portion carried by the subbase/subgrade, and the portion carried by the load transfer mechanism. These components have to be in equilibrium, and during the measurement of LTE, if one carries more of the imposed load, the others will carry less.

VOLUMETRIC SURFACE TEXTURE

In an attempt to establish a method of quantifying the decrease in load transfe

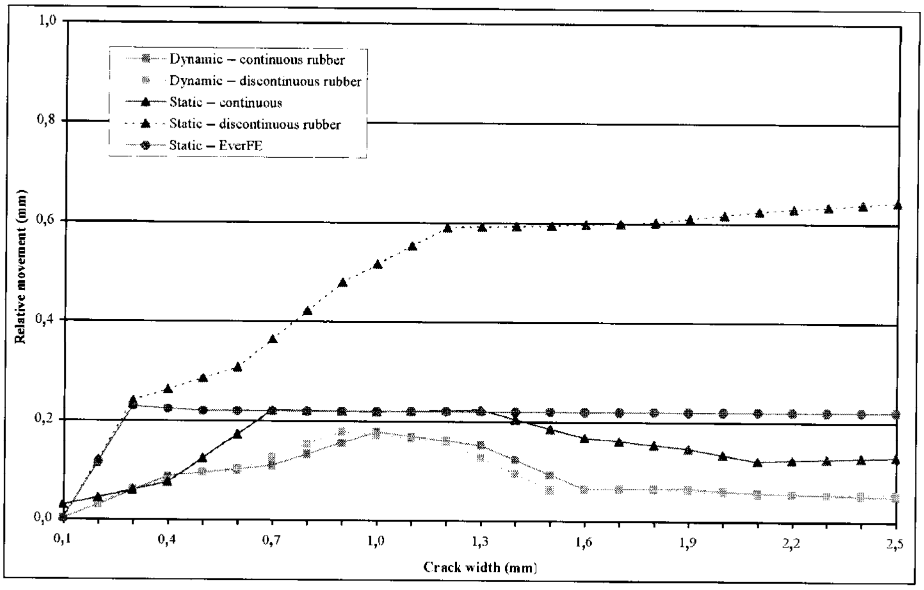


Figure 4 Relative movement versus crack width - comparison between discontinuous and continuous rubber foundation (pre-deformed plastic joint)

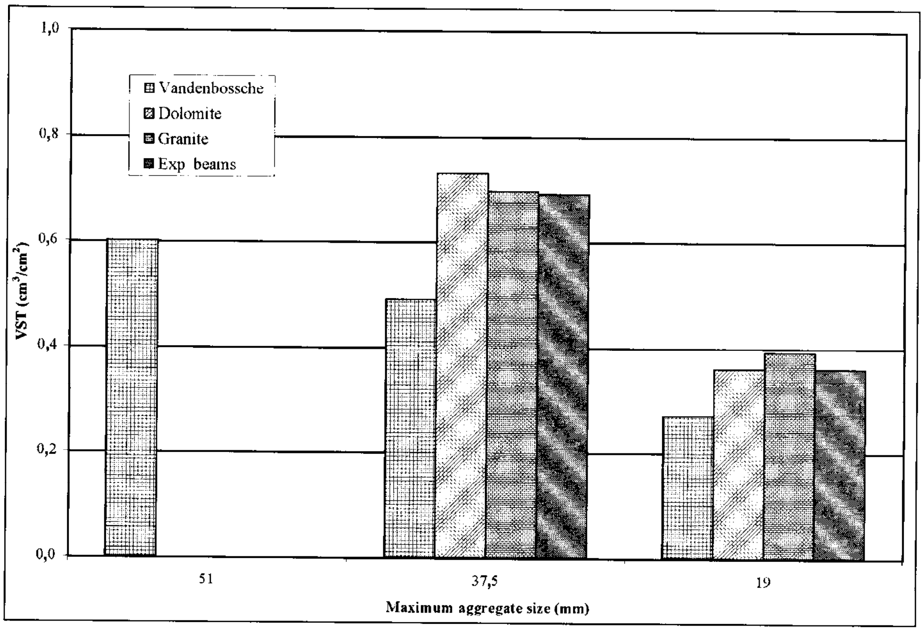


Figure 5 Effect of maximum coarse aggregate size on VST - comparison between USA and South African aggregates

efficiency with an increase in crack width, and to provide an estimate of the aggregate wear-out that has taken place since fracture, Vandenbossche (1999) developed a volumetric surface texture (VST) test at the University of Minnesota.

Dolomite and granite sand with approximately the same grading was used for both the 19 mm and 37,5 mm coarse aggregate concrete mixes. This was to obtain equivalent crack surfaces for both 19 mm and 37,5 mm aggregate size concrete slabs respectively. A computerised workshop milling machine was modified to measure the VST of representative samples of each of the first four experiments.

The VST of the crack faces of the experimental slabs, as well as crack faces formed during modulus of rupture testing of concrete slabs, was determined to

- quantify whether the crack face volumes formed by the 19 mm granite and dolomite aggregate were indeed similar, and also for the 37,5 mm granite and dolomite aggregate
- quantify whether the crack faces formed by breaking 28-day slabs during modulus of rupture testing could be taken as representative of the crack surface in the experimental slabs – in other words, whether it may only be necessary to cast a slab, break it and measure the VST, to determine the VST of a crack inside a pavement
- compare the VST results from this study, obtained using the high-quality South African crushed aggregates, with the results obtained from crack faces of concrete constructed with typical USA aggregates, published by Vandenbossche (1999)

The results published by Vandenbossche (1999) based on VST measurements made with cores from 16 different dowelled contraction joints are reproduced in figure 5, together with the results obtained in this study. Although the joints considered in this study were aggregate interlock joints, the volume of the aggregate interlock crack face itself could still be compared.

From figure 5 it is clear that the VST of the 19 mm granite and dolomite crack face volumes, as well as the 37,5 mm granite and dolomite crack face volumes, differed less than 10 % from each other. Furthermore, the VST of the crack faces of the 19 mm and 37,5 mm experimental slabs were also approximately the same as the crack faces formed by breaking 28-day slabs during modulus of rupture testing. It can therefore be accepted that the crack face formed when conducting modulus of rupture testing on 28-day test slabs can be taken as representative of the VST of the crack inside the road pavement.

When comparing the VST results from this study with the results obtained from crack faces of concrete constructed with typical USA aggregates, published by Vandenbossche (1999), it is clear that the South African aggregates have a greater aggregate interlock potential. The VST results obtained for the 19 mm and 37,5 mm maximum aggregate size concrete were 37 % and 44 % higher than the USA results, respectively.

IMPROVEMENT OF AGGREGATE INTERLOCK EQUATION

Reference has been made to the fact that the results of the research presented here formed part of the overall process of producing a new mechanistically based concrete pavement design method for South Africa. The software package by the name of Cncpave that was developed still contained an empirical equation (developed through multiple regression analyses after full-scale experiments) for calculating the aggregate interlock factor, Ca (Strauss *et al* 2001). In order to replace the empirical equation in the source code of the software package it was necessary to develop an equation from the laboratory studies covering the range of data measured. The equation is as follows:

$$y(x) = 0{,}118\left(1 - e^{-\left(\left(v + \frac{11{,}413}{agg}\right)x\right)^{1{,}881}}\right) \qquad (2)$$

Where:
$y(x)$ = Relative vertical movement at crack/joint (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)

The relative vertical movement values

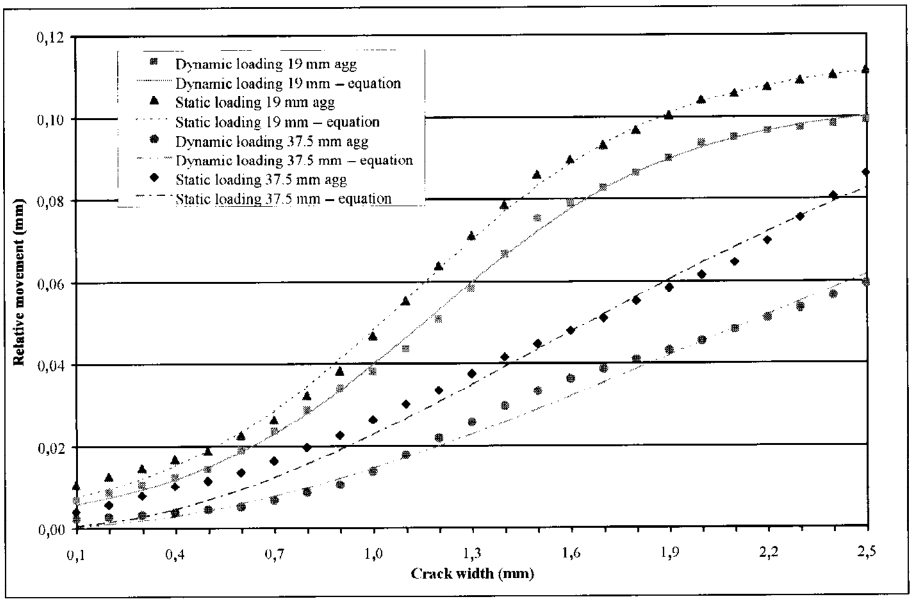
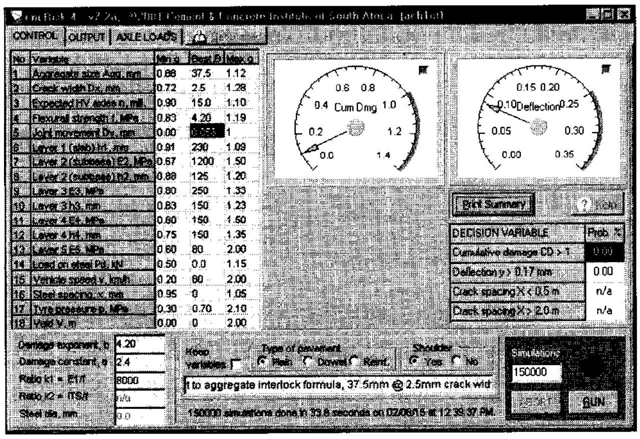


Figure 6 Comparison between measured values and values predicted with equation 2



No	Variable	Min α	Best β	Max α
1	Aggregate size Agg, mm	0.88	37.5	1.12
2	Crack width Dx, mm	0.72	2.5	1.28
3	Expected HV axles n, mil.	0.90	15.0	1.10
4	Flexural strength f, MPa	0.83	4.20	1.19
5	Joint movement Dv, mm	0.00	[illegible]	1
6	Layer 1 (slab) h1, mm	0.91	230	1.09
7	Layer 2 (subbase) E2, MPa	0.67	1200	1.50
8	Layer 2 (subbase) h2, mm	0.88	125	1.20
9	Layer 3 E3, MPa	0.80	250	1.33
10	Layer 3 h3, mm	0.83	150	1.23
11	Layer 4 E4, MPa	0.80	150	1.50
12	Layer 4 h4, mm	0.75	150	1.35
13	Layer 5 E5, MPa	0.60	80	2.00
14	Load on steel Pd, kN	0.50	0.0	1.15
15	Vehicle speed v, km/h	0.20	60	2.00
16	Steel spacing x, mm	0.95	0	1.05
17	Tyre pressure p, MPa	0.30	0.70	2.10
18	Void V, m	0.00	0	2.00

DECISION VARIABLE	Prob %
Cumulative damage CD > 1	0.00
Deflection y > 0.17 mm	0.00
Crack spacing X < 0.5 m	n/a
Crack spacing X > 2.0 m	n/a

Damage exponent, b: 4.20
Damage constant, a: 2.4
Ratio k1 = E1/f: 8000
Ratio k2 = ITS/f: n/a
Steel dia, mm: 0.0

Keep variables ☐
Type of pavement: ● Plain ○ Dowel ○ Reinf.
Shoulder: ● Yes ○ No
Simulations: 150000

t to aggregate interlock formula, 37.5mm @ 2.5mm crack wid

150000 simulations done in 33.8 seconds on 02/08/15 at 12:39:37 PM.

Figure 7 Input detail for main control panel

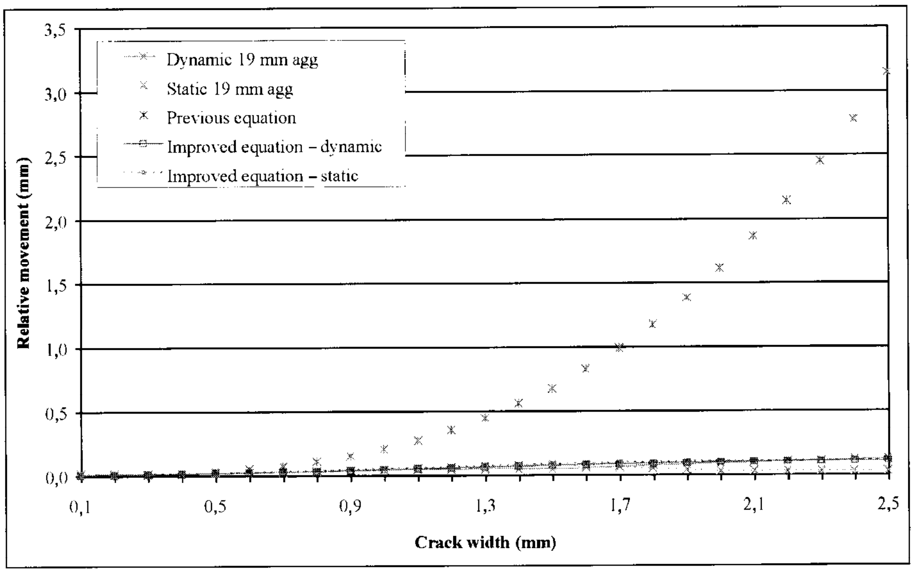


Figure 8 Improvement in relative movement calculated with equation 2

measured during the experiments were compared with the theoretical values calculated with equation 2 (see figure 6). The sum of the squares of the errors between the measured and the calculated values was as follows:

- Dynamic loading – 19 mm aggregate: 0,24
- Static loading – 19 mm aggregate: 0,21
- Dynamic loading – 37,5 mm aggregate: 0,12
- Static loading – 37,5 mm aggregate: 0,16

The original version of the software containing the empirical equation was used to conduct so-called 'before' analyses. For the input on the main control panel of the software, typical jointed concrete pavement parameters were used (see figure 7). 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.

Equation 2 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 best illustrated by the shift in the results obtained with the original equation to the results obtained with the improved equation (see figure 8).

From figure 8 it is clear that at a crack width of 2,5 mm, the equation previously used gave results 10 times higher than what was physically measured in the laboratory for especially the 19 mm coarse aggregate concrete mixes.

CONCLUSIONS

The main conclusions reached after interpretation of experimental results were as follows:

- An increase in crack width caused an increase in deflection, a decrease in deflection load transfer efficiency, and an increase in relative movement.
- The larger 37,5 mm coarse aggregate had lower deflections and greater deflection load transfer efficiencies than the smaller 19 mm coarse aggregate at the same crack widths during dynamic and static loading.
- The deflection measurements tended to converge at crack widths larger than 2,5 mm, indicating that at this crack width the stiffness of the subbase governed the results obtained.
- For the same maximum size coarse aggregate, the load transfer efficiency was larger where there was a continuous rubber support (rubber not cut through) than where there was a crack simulated in the subbase (top rubber layer cut through).
- The software package EverFE can be used to give an indication of the results that could be experienced in practice, but it is not suited to South African aggregates and climatic conditions.
- The volumetric surface texture (VST) ratio of the 19 mm granite and dolomite crack face volumes, as well as the 37,5 mm granite and dolomite crack face, differed less than 10 % from each other.
- The VST of the crack faces of the 19 mm and 37,5 mm experimental slabs were approximately the same as the crack faces formed by break-

ing slabs during modulus of rupture testing. It can therefore be accepted that the crack face formed when conducting modulus of rupture testing on 28-day test beams (750 x 150 x 150 in size) can be taken as representative of the VST of the crack inside the road pavement.

- When comparing the VST results from this study with the results obtained from crack faces of concrete constructed with typical US aggregates, published by Vandenbossche (1999), it is clear that the South African aggregates have a greater aggregate interlock potential. The VST results obtained for the 19 mm and 37,5 mm coarse aggregate size concrete were 37 % and 44 % higher then the US results, respectively.
- The main contribution to the current state of knowledge was the development of a mechanistic equation quantifying the effect of aggregate interlock at a joint/crack in a concrete pavement. This equation has already been included and tested in the Cncpave software package developed as part of the upgrading of the new mechanistic concrete pavement design method for southern Africa.

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Note: Hanekom, A C (married name) is the same entity as Brink, A C (maiden name and present title).