# Ultra Thin Continuously Reinforced Concrete Pavement Research in South Africa

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ABSTRACT: Ultra thin continuously reinforced concrete pavements (UTCRCP), in literature also referred to as Ultra Thin Heavy Reinforced High Performance Concrete (UTHRHPC), have been used in Europe successfully as a rehabilitation measure on steel bridge decks and reported on at the 5th International CROW workshop in Istanbul (2004). This concept has been explored further in South Africa by constructing experimental sections of 50 mm UTCRCP directly on top of both natural gravel and cement-treated natural materials. The sections were tested using the Heavy Vehicle Simulator (HVS) and rendered structural lives that varied from 5 million to 90 million equivalent standard (80 kN) axles. Distress that developed indicated sensitivity to the bond between UTCRCP and the support, thickness of the UTCRCP layer, concrete strength, the development of cracks and the presence of water. In order to further explore the applicability of this concept under different conditions, use was made of 3D-finite element modelling of a pavement under a moving load. The thickness of UTCRCP layer, placed on top of a support system with varying stiffness, was varied from 40mm to 240mm. Varying degrees of bonding, the presence of voids, varying concrete properties as well as the position of and different quantities of steel reinforcement were also modelled. The paper discusses the similarity of pavement response between HVS loading and modelling.

KEY WORDS: Ultra Thin Continuously Reinforced Concrete Pavement (UTCRCP), Ultra Thin Heavy Reinforced High Performance Concrete (UTHRHPC), High Performance Concrete, Ultra Thin White Topping, Finite Element Modelling, Heavy Vehicle Simulator (HVS)

# 1. INTRODUCTION

With more than 70 % of the road network in South Africa older than its 20-year design life, major investments into structural strengthening (Capex) works is required. Due to severe budget constraints, new innovative pavement repair strategies need to be developed that will be more cost effective. The requirements for such an innovative pavement repair strategy are:

- Must be able to be applied to an existing road surface with minimal preparation works required to the existing road surface or structures;
- Must be able to be constructed with existing road construction equipment currently available in South Africa;
- Must be able to be opened to traffic within 48 hours – on a roads carrying in excess of 100,000 vehicles per day a major logistical problem is the accommodation of traffic during construction;
- Must have structural life expectancy in excess of 30 years with minimal maintenance requirements during this period – indicating an ability to successfully withstand increased axle loads and tyre pressures of modern heavy vehicles;
- Must be able to and meet all requirements to ensure a safe road surface under all conditions, and
- Must be cost affective.

One such a possible innovative pavement repair strategy identified is Ultra Thin Continuously Reinforced Concrete Pavements (UTCRCP), in literature also referred to as Ultra Thin Heavy Reinforced High Performance Concrete (UTHRHPC). UTCRCP have been utilised as an industrial floor solution in Europe, and recently in the rehabilitation of steel bridge decks in the Netherlands. The UTCRCP layer design consisted of 8 mm diameter welded deformed steel bar mesh placed at 50 x 50mm intervals and embedded at the neutral axis in an Ultra High Strength Cement (UHSC) concrete paste as illustrated in Figure 1. The concrete paste consisted of normal aggregate (6.75mm stone) mixed with portland cement (CEM I 42.5), steel- and polypropylene fibres and an ultra-fine filler (0.1 – 0.2 µm size with specific surface area of 25 000 m2/kg). A low water:cement ratio of 0.28 to 0.3 was used to enable the mix to develop sufficient strength allowing the pavement to be opened to traffic within 48 hours after placement. The paste has a packing density of 0.7 – 0.9 (similar to ceramic & glass). It is less porous than normal concrete and produces a much greater bond between steel reinforcement and the concrete. Bond to a 6mm smooth steel bar is reportedly 4 to 5 times greater than normal concrete. The basic material performance properties reported by Contec-APs (Buitelaar, 2004) of the UTCRCP against that of normal CRCP are shown in Table 1. Finite element modelling as well as laboratory studies together with field applications have been reported on at the 5$^{th}$ International CROW workshop in Istanbul by Braam et al (2004).

**Figure 1: Typical UTCRCP Example (Contec Ferroplan$^{TM}$)**

**Table 1: UTCRCP performance properties with steel fibers (without bar reinforcement)**

| Description | CRCP | UTCRCP |
|---|---|---|
| Compressive Strength (MPa) | 35 to 40 | 120 to 140 |
| Flexural Strength (MPa) | 4.2 to 4.5 | 7 to 15 |
| Joints | Longitudinal | Construction |
| Opening Time to Traffic | 7 to 14 days | 24 Hours |

The use of UTCRCP for the structural strengthening of roads has been explored further in South Africa by constructing experimental sections with 50 mm UTCRCP, initially using the European specifications for concrete mix and reinforcement (Buitelaar, 2004). The sections were subsequently tested using the Heavy Vehicle Simulator (HVS) (Steyn et al, 1999), the results obtained were then evaluated through 3D-finite element modelling and the design was refined using laboratory tests. The refined UTCRCP design was then subjected to further HVS testing. The rest of the paper discusses the above process in more detail.

## 2. UTCRCP PHASE 1: EVALUATION OF CONTEC FERROPLAN$^{TM}$

### 2.1 Background

The main objective of phase 1 of testing was to evaluate the potential of Contec Ferroplan$^{TM}$ as a pavement layer under Accelerated Pavement Testing (APT) using the Heavy Vehicle Simulator (HVS). Since the performance of a pavement layer is influenced by various factors which include traffic loading, substructure support, environment, design, construction and maintenance, our experiment had to evaluate the sensitivity of the UTCRCP for as many of these factors as possible. The experimental design consisted of the following two phases:

- Accelerated Pavement Testing (APT): Two experimental sections each 60m long and 3,7m wide was constructed to simulated three-substructure support strengths, i.e.
  - weak (1.2mm average maximum deflection @ 40kN) obtained by ripping and re-compacting insitu material;
  - medium (0.6mm average maximum deflection @ 40kN) obtained by cement stabilising (4.0% CEM II 32.5 A-L) 150mm of insitu material , and
  - strong (0.3mm average maximum deflection @ 40kN) obtained by cement stabilising (4.0% CEM II 32.5 A-L) 300mm of insitu material.

  The design thickness of UTCRCP UTCRCP layer was 50mm, the design mix as proposed by Contec-APs (Buitelaar, 2004) was used and the mix was placed using an asphalt paver. These sections were subject to various accelerated tests using the Heavy Vehicle Simulator (HVS).

- Long Term Performance Phase (LTPP): For this phase, the existing screener lanes at Heidelberg Traffic Control Centre (HTCC) was overlaid using an asphalt paver with 50 mm UTCRCP to the same design as short term test sections. The traffic control centre screener lanes provided us with accurate axle measurements for every heavy vehicle that passed over the section.

## 2.2 UTCRCP Layer Construction

In-line with the innovative pavement repair requirement that it must be able to be constructed with existing road construction equipment currently available in South Africa, the 50mm UTCRCP layer was placed using a normal asphalt paver. This required the paver to travel on top of the reinforcement mesh. In order to keep the reinforcing mesh at the correct height (close to the neutral axis) under the weight of the paver the following measures were employed:

- 10mm diameter steel bar spacers were longitudinally placed at 200mm intervals on top of the AC layer within the paver wheelpath as illustrated in Figure 2;
- 10mm diameter steel rods 150mm in length were placed vertically at 2 meter intervals along the internal area of the mesh to act as anchors, and
- 10mm diameter steel rods 500mm in length were placed at 45º angle at 2m intervals along the edges of the mesh to limit curling under weight of paver and during subsequent curing. To investigate the effectiveness of the edge anchors, an area of approximately 10m in length was not anchored at all.

In addition to the benefits of the steel anchors during construction it was also anticipated that the anchors will restrict the vertical debonding between the concrete, asphalt and base coarse during the life cycle of the pavement.

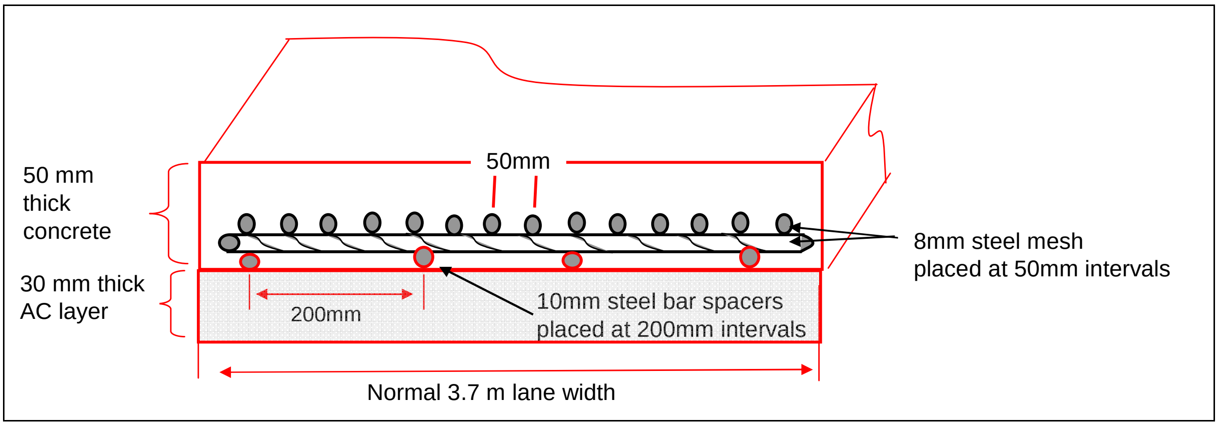



**Figure 2: UTCRCP Overlay System – Phase 1**

## 2.3 UTCRCP Accelerated Pavement Testing

### 2.3.1 HVS Testing


Six HVS tests were conducted on the two experimental sections to evaluate the performance of UTCRCP under the influence of the following variables:

- Loading: normal wheel path loading vs edge loading (most aggressive). All tests were done with channelized traffic (no wander) with the HVS in the bi-directional trafficking mode (trafficking in both directions);
- Environmental influences on UTCRCP layer curl and warping;

- UTCRCP performance on top of various degrees of sub structure support (strong / medium / weak / and even no support at all);
- UTCRCP performance under the influence of surface water;
- UTCRCP performance under a bitumen-rubber chipseal and surface water, and
- The performance of longitudinal and transverse construction joints under the influence of accelerated trafficking.

Figure 3 show the layout of the HVS tests on the two experimental sections and along with Table 2 give some details of the parameters being varied as well as details of the HVS testing sequence. The HVS test pad was 1m wide and 8m long as shown. In the channelized traffic mode, the test pad width was reduced to the total width of a dual wheel which is approximately 650mm wide. The HVS applied approximately 18 000 bi-directional load repetitions per day. To investigate the effects of surface water on the formation of cracks, de-bonding and cavities under the concrete, surface water was introduced at certain stages of testing. Prior to the start of the abovementioned HVS tests, the influence which daily temperature variations had on the curling and warping characteristics of the concrete were also investigated.

### 2.3.2 HVS Data Collection Instrumentation

The following instrumentation was used to evaluate the performance of the various test sections under the influence of the environment and accelerated loading 24-hours a day, seven days a week:

- Multi-depth Deflectometers (MDDs) to measure the surface and in-depth elastic and plastic movements of layers;
- Joint Deflection Measuring Devices (JDMDs) to measure the surface deflections and permanent deformation at various locations along the load path as well as the longitudinal edge;
- Thermocouples to measure air, surface and in-depth pavement temperatures;
- Weather station to measure air temperatures, wind speed & direction, relative humidity and precipitation;
- Instrumentation to measure the HVS loading: magnitude, number of repetitions and wheel speed, tyre pressure, and
- High density photography to investigate the appearance of cracks, crack development and crack widths.

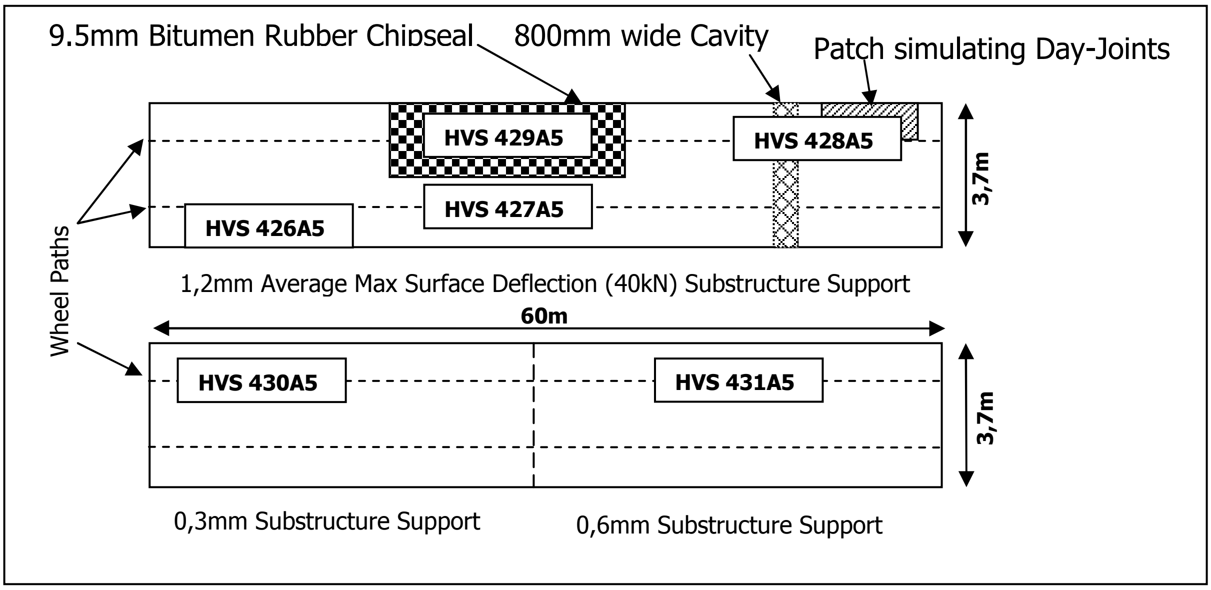
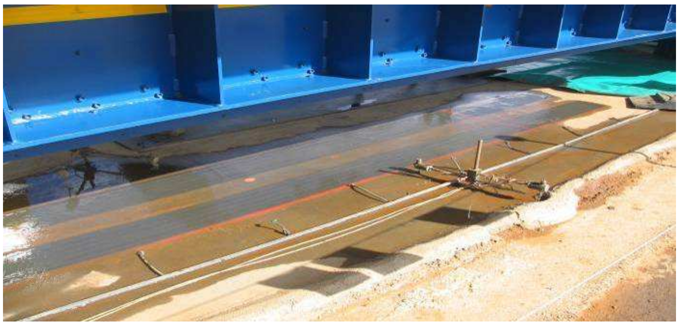



**Figure 3: HVS Test Layout on Experimental Sections – Phase 1**

**Table 2: UTCRCP HVS Test Overview – Phase 1**

| HVS Test No | Test Description |
|---|---|
| HVS 426A5  | 140kN Aircraft Wheel Loading on edge of UTCRCP layer, no visible shrinkage cracks, base with 1.2mm surface deflection @ 40kN, surface water added for last 103,000 repetitions.<br><br>AIM: Establish whether UTCRCP can achieve minimum of 50 million E80's on insitu earth material. |
| HVS 427A5  | 80kN Dual Wheel Loading in wheel path, visible shrinkage cracks, base with 1.2mm surface deflection @ 40kN, surface water added.<br><br>AIM: Establish impact of continuos surface water on UTCRCP performance and pumping of base material. |
| HVS 428A5  | 80kN Dual Wheel Loading on joints in wheel path and over 800mm wide cavity, base with 1.2mm surface deflection @ 40kN, UTCRCP has "day" joints, surface water added.<br><br>AIM: Establish the performance of joints within UTCRCP as well as performance of UTCRCP in complete absence of substructure support. |
| HVS 429A5  | 80kN Wheel Loading in wheel path, visible shrinkage cracks in UTCRCP covered with 9.5 mm Bitumen Rubber Chipseal, base with 1.2mm surface deflection @ 40kN, surface water added.<br><br>AIM: Establish adhesion of Bitumen Rubber Chipseal to UTCRCP and its ability to reduce surface water ingress |

**Table 2: HVS Test Overview - Continued**

| HVS Test Description | Repetitions |
|---|---|
| HVS 430A5  | 80kN Wheel Loading in wheel path, visible shrinkage cracks, base with 0.3mm surface deflection @ 40kN, surface water added, wheel load increased to 140 kN for last 40,000 repetitions.<br><br>AIM: Establish performance impact of strong substructure support on UTCRCP |
| HVS 431A5  | 80kN Wheel Loading in wheel path, base with 0.6mm surface deflection @ 40kN, visible shrinkage cracks, surface water added.<br><br>AIM: Establish performance impact of medium substructure support on UTCRCP |

## 2.4 UTCRCP APT RESULTS

### 2.4.1 Load Repetitions

The main objective of this phase of testing was to evaluate the potential of UTCRCP under Accelerated Pavement Testing (APT) using the Heavy Vehicle Simulator (HVS). The various HVS loading regimes for the different tests are shown in Table 3. To investigate the bond and potential development of cavities under the concrete UTCRCP layer, surface water was added during testing.

As seen from the results of HVS Test 426A5, the achieved life of the CONTEC FERROPLAN® layer of 92 million E80's exceeded all expectations in the dry state under a 140 kN wheel load or in other terms a 28 ton axle load. Was it not for the introduction of surface water, the layer would not have failed. Based on these results the decision was made that all subsequent HVS Phase 1 tests will be conducted with the continuous addition of surface water, since the CONTEC FERROPLAN® layer seemed to have a perpetual life under dry conditions. As seen form the subsequent test results much shorter lives was achieved under continuous addition of surface water. At first glance these results might seem to indicate complete failure of the system when compared to the first test. If one however considers that i.e. the results for Test 430A5 is equivalent to 15 years of traffic on the N3 with the road surface being wet for each and every day, one appreciates the potential of the Ultra Thin Continuously Reinforced Concrete Pavement (UTCRCP) solution.

**Table 3: Loading regime for each HVS test**

| Test Wheel Load (kN) | HVS Repetitions Per Test | | | | | |
|---|---|---|---|---|---|---|
| | 426A5 | 427A5 | 428A5 | 429A5 | 430A5 | 431A5 |
| 30 | | | | 2,000 | | |
| 40 | 18,000 | 40,000 | 100,000 | 38,000 | 40,160 | 40,000 |
| 60 | 20,500 | 40,000 | 40,500 | 40,000 | 40,080 | 40,088 |
| 80 | 26,500 | 197,000 | 59,650 | 177,704 | 122,764 | 120,000 |
| 80 | | 130,500 | | | | |
| 80 | | 122,500 | 52,650 | | 800,000 | 703,189 |
| 100 | 23,000 | | | | 21,547 | |
| 120 | | | | | 20,528 | |
| 125 | 137,000 | | | | | |
| 140 | 135,100 | | | | | |
| 140 | 103,300 | | | | 1,360 | |
| Total Reps | 463,400 | 530,000 | 252,800 | 257,704 | 1,046,439 | 903,277 |
| Total E80's (n=4.5) | 92,190,469 | 10,470,348 | 2,892,170 | 4,306,993 | 25,761,052 | 18,915,197 |

Notes:

1...\ The shaded areas in the table indicate the periods during which surface water was continuously added.

2...\ The total amount of repetitions in the table indicates the amount of HVS repetitions (Please Note: E80 = $(\text{Load}/40)^{4.5}$ since it is wheel load and not axle load) to failure.

3...\ An aircraft wheel was used for wheel loads above 100kN

### 2.4.2 Surface Deflection

The 40kN surface deflections against the number of repetitions of all the tests can be seen in Figure 4. The figure displays the maximum deflection per test as captured by the JDMDs.

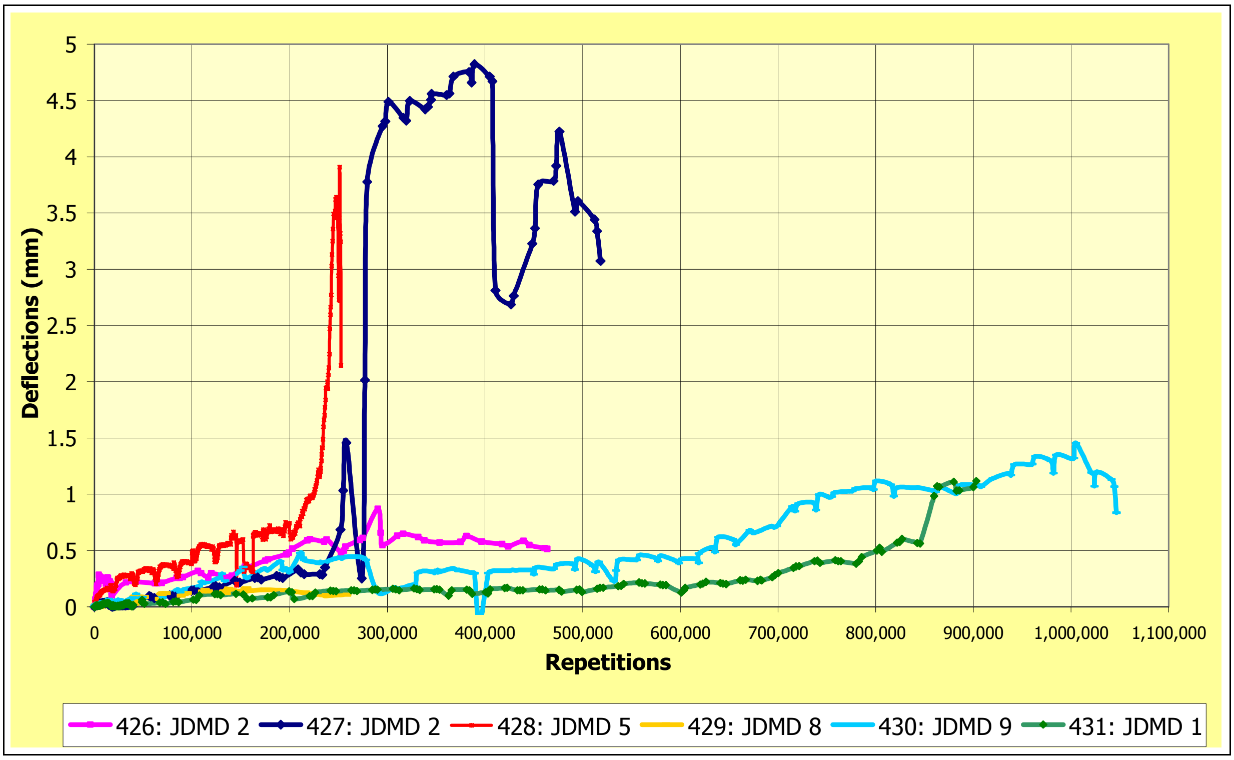



**Figure 4: Peak 40kN Surface Deflections**

Comparing the high load test which was conducted at the edge of the pavement 426A5 with the test at the interior of the UTCRCP layer where water was added right from the start 427A5, it is obvious that water and the subsequent formation of a cavity due to pumping caused deflections significantly higher than what was recorded during the edge load test 426A5. The effect of curling also seemed to be of low significance. The cemented base layer section seems to provide better protection than the granular test section against the formation of voids between the concrete and sub-layers under the influence of water. Significantly lower deflections in sections 430A5 and 431A5 were recorded in comparison with sections 426A5 and 427A5.

### 2.4.3 Surface Permanent Deformation

The permanent deformations as recorded by the same instruments as above are shown in Figure 5. The same conclusion is drawn regarding the permanent deformation of the pavement sections. Test 427A5, which was exposed to water from the start of the test, had by far the biggest amount of permanent deformation. This observation was confirmed with cores and slots which were cut after the completion of the test. It is also important to note that a dramatic increase in permanent deformation occurred at the end of the test immediately before failure.

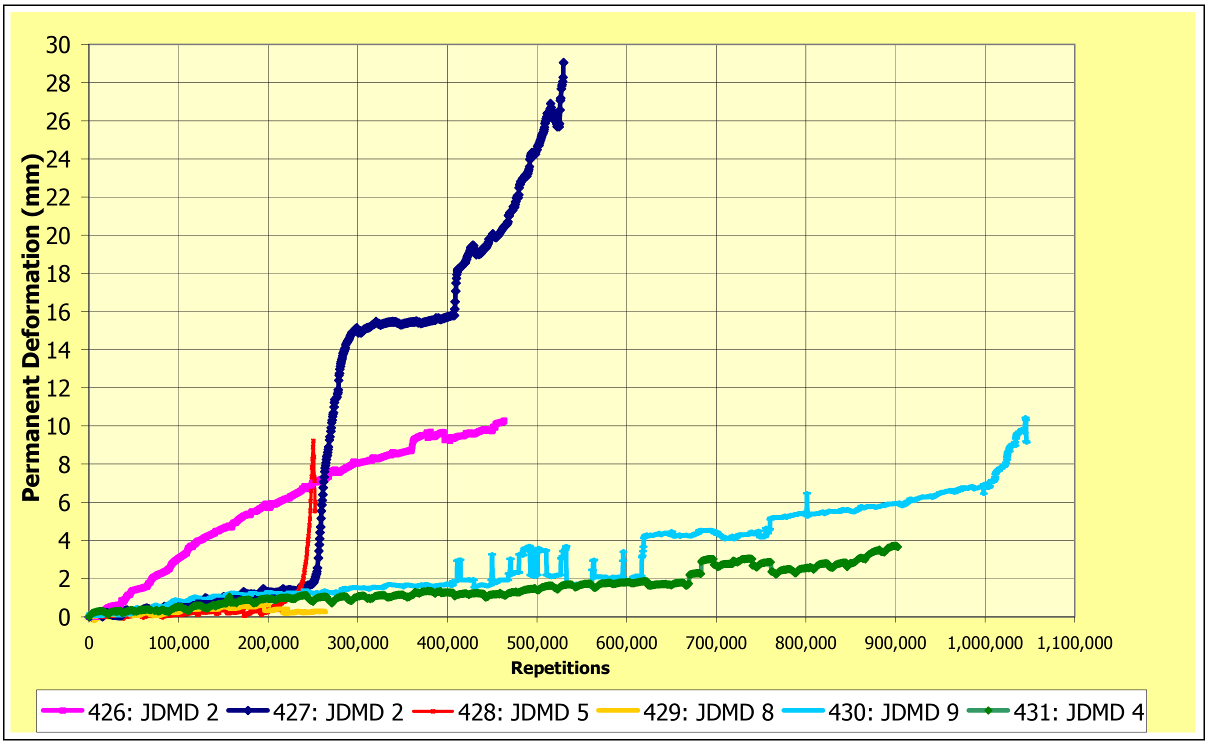



**Figure 5: Surface Permanent Deformation**

### 2.4.4 Visual Deterioration

Cracks and crack growth were monitored on a daily basis on all tests. Failure generally occurred where a transverse crack had developed due to shrinkage. After the initial crack, cracks developed parallel to the direction of loading but approximately 300 mm from the loaded area. Subsequently, circular shaped cracks developed around the first transverse crack and these rapidly developed into wider cracks with the addition of water. Cores drilled after the completion of the tests revealed a loss in bond between the concrete and the substructure under the loaded area and the formation of a cavity at the failed areas. With additional trafficking, the concentration of the cracks intensified to eventually form concentric circles around the area of final failure which occurred almost instantaneously after the formation of these cracks.  Table 4 shows the typical crack patterns observed at the failed area for the various tests.

**Table 4: HVS Test Section Visual Failure Photo's**

<table>
<tr><td>HVS Test 426A5<br></td><td>HVS Test 427A5<br></td></tr>
<tr><td>HVS Test 428A5<br></td><td>HVS Test 429A5<br></td></tr>
<tr><td>HVS Test 430A5<br></td><td>HVS Test 431A5<br>
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## 2.5 HVS Phase 1 Field Test Conclusions

From the detailed diagnostic analysis of the failures that occurred on the HVS test sections a pattern emerged that indicated to problems related with the mix design, and associated construction of the layer. Problem areas identified include:

- **Cement Type**: Although CEM I cement was used, subsequent tests on CEM I products from three producers indicated performance differences of up to 50 %. The CEM I used in UTCRCP layers was the worst performing;
- **Material Mixing**: The mixing of pavement materials was done on site using a Ready Mix truck, as a result of the nature of the mix i.e. low water content and steel fibres the mixing action of the truck was not able to induce sufficient mixing energy to properly mix the materials;
- **Layer Placement**: The maximum aggregate used in UTCRCP was 6.75mm, this in combination with the fact that the layer was placed with an Asphalt paver, requiring very low slump, and the high steel mesh content resulted in steel mesh not being completely covered by the cement paste;
- **Steel Fibre Type**: The drawn wire steel fibres used proved to be incorrect in terms of length and shape, contributing to the rapid shear failures (or popping) observed in the top of the UTCRCP layer, and
- **Steel Bar Spacers**: The 10mm diameter steel bar spacers that were longitudinally placed at 200mm intervals on top of the AC layer to support the asphalt paver, acted as crack inducers and subsequent laboratory tests identified them as one of the major contributors to failure of the UTCRCP layer.

To address these identified problem areas, the decision was made to make use of a combination of laboratory tests and Finite Element Modelling at the initiation of phase 2. Not only are the above procedures cheaper and quicker than HVS field tests, but more importantly the variables can be much better controlled.

# 3. UTCRCP PHASE 2 - SOUTH AFRICAN DESIGN

## 3.1 Theoretical Model Development

### 3.1.1 Finite Element Model

The purpose of the Finite Element (FE) modelling was to firstly develop a model that could predict the failures that was observed during phase 1 HVS testing, and secondly be used to investigate the relative effect of the following parameters on UTCRCP performance:

- amount and position of the steel mesh reinforcement;
- thickness and stiffness of the UTCRCP layer;
- bond and the effect of a void between UTCRCP layer and the supporting substructure;
- stiffness of the supporting substructure, and
- speed of loading.

The UTCRCP FE model that was developed consisted of a thin highly-reinforced concrete layer on top of a layered support system. The properties of the different components are listed in Table 5.

**Table 5: UTCRCP FE Model Component Properties**

| Layer | Material | Thickness (mm) | Modulus (MPa) | Density (kg/m$^3$) |
|---|---|---|---|---|
| 1 | Soil | 1,350 | 50 | 1,500 |
| 2 | Subbase | 300 | 150 | 1,800 |
| 3 | Base | 145 | 300 to 1,200 | 1,900 |
| 4 | Interlayer | 5 | 0 to 1,200 | 1,900 |
| 5 | UTCRCP Concrete | 40 to 240 | 50,000 to 80,000 | 2,400 |
| 6 | UTCRCP Reinforcement | 0 to 4% | 200,000 | 7,850 |

To develop the UTCRCP FE model of a representative piece of road the following assumptions and procedures were utilised:

- The mesh of the model per layer was selected to be finer at the top and coarser at the bottom in order to reduce computational effort. The finest mesh was directly in the wheel path. At both ends of the wheel path, for a distance of approximately 1m, the mesh was also coarser, basically allowing for the wheel speed to increase to a constant value in these areas;
- The boundary conditions were such that at the bottom, all displacements (x, y and z directions) were fixed. On the plane of symmetry at y=0, the displacement in the y direction was constrained to zero. At the ends as well as along the side, non-reflective boundaries were prescribed, meaning that pressure waves that propagated radially outwards from the wheel positions were not reflected back into the modelled section;
- Two circular load patches, 200mm diameter, simulated the double wheel loading of a typical truck. A surface pressure of 700 kPa was applied to the top concrete surface along a path that ran at a distance of 900 mm from the plane of symmetry;
- In the dynamic analysis the wheel load, together with the gravity load of the concrete was first applied in 0.1 seconds from 0 to 700 kPa at an axle position close to the one end of the model (145 mm). Thereafter it was accelerated for a distance of 855 mm along the path up to the ultimate speed. Thereafter the wheel load moved at a constant speed along the wheel path;
- A transverse crack in the concrete was modelled at the centre of the wheel path over the full width of the model. The crack was through the total thickness of the concrete and was modelled in such a way that compression forces but no tension forces could develop on the two adjacent surfaces of the crack. Vertical shear could also be carried in the crack, which meant that there was no vertical slip between the surfaces of the crack. The reinforcement was modelled as a continuous sheet of steel in both the x and y directions;
- In order to create a void below the UTCRCP layer, the interlayer stiffness was reduced to practically zero, while for transition areas at both ends of the void area, reduced stiffnesses were introduced by linear interpolation;
- A non-linear dynamic analysis was required since the wheel patches were moving objects in the wheel path and sliding contact is defined between the wheels and the concrete surface. At the simulated crack in the concrete, non-linear contact was also defined between the surfaces of the crack, and
- The results of the analyses were available after a run as time history displacements, strains as well as stresses. These could be presented as time history plots or fringe or contour plots at selected time instances as seen in Figure 6. Deformed plots scaled to enlarge the actual deformation were also generated.

In view of the time and cost of using sophisticated FE analysis, a limited number of cases were evaluated. The data generated by the FE analysis was used to re-calibrate the equations used in cncPave. cncPave was subsequently used as a tool to illustrate the relative effect of different parameters on the performance of the UTCRCP. Figure 7 shows the reliability of the predicted maximum tensile stress on the surface of the pavement using the cncPave equation, compared to the values generated by FE analysis ($R^2$ = 0.91).

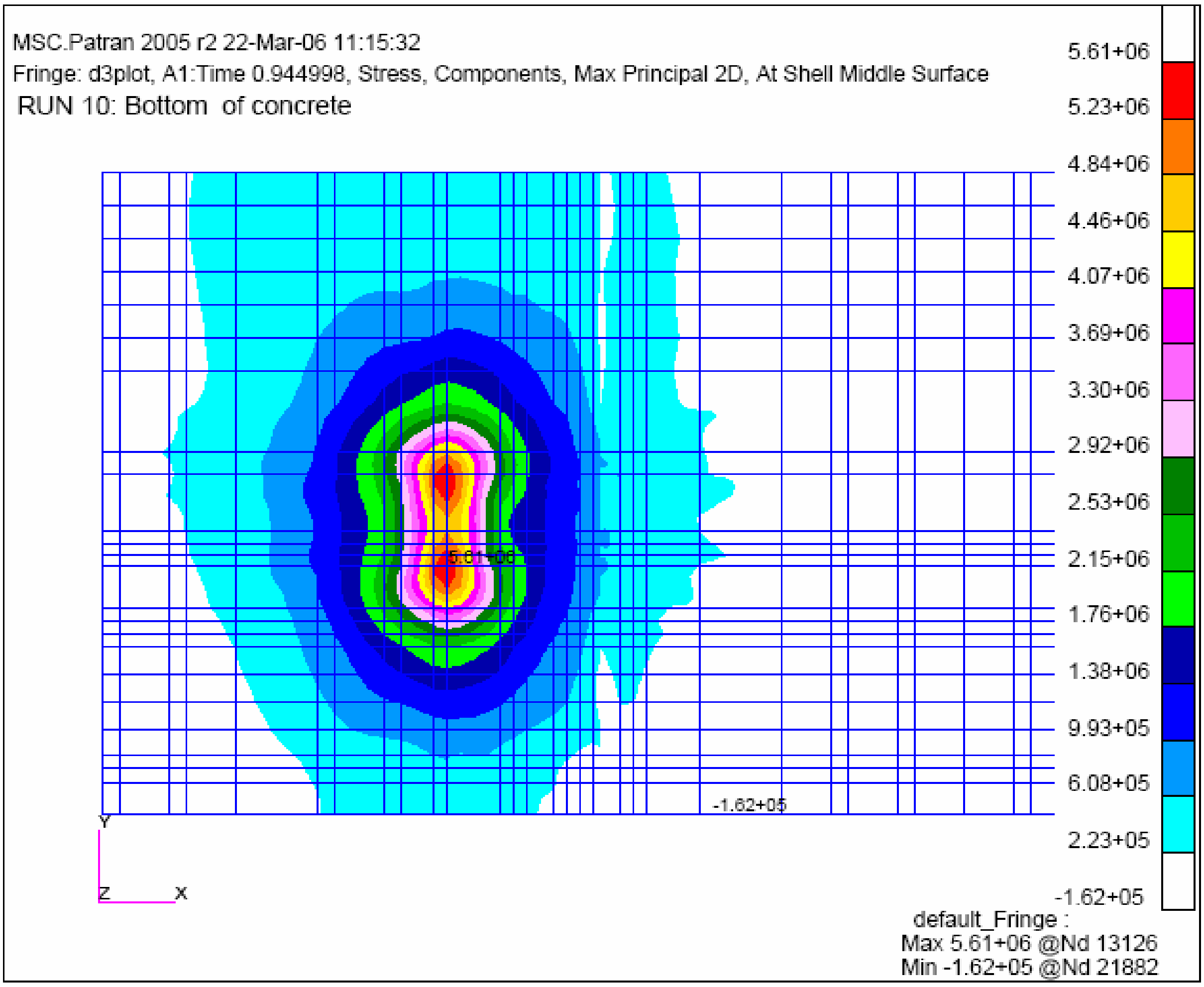



**Figure 6: Typical Contour Plot of Maximum Stress at Bottom UTCRCP Layer**

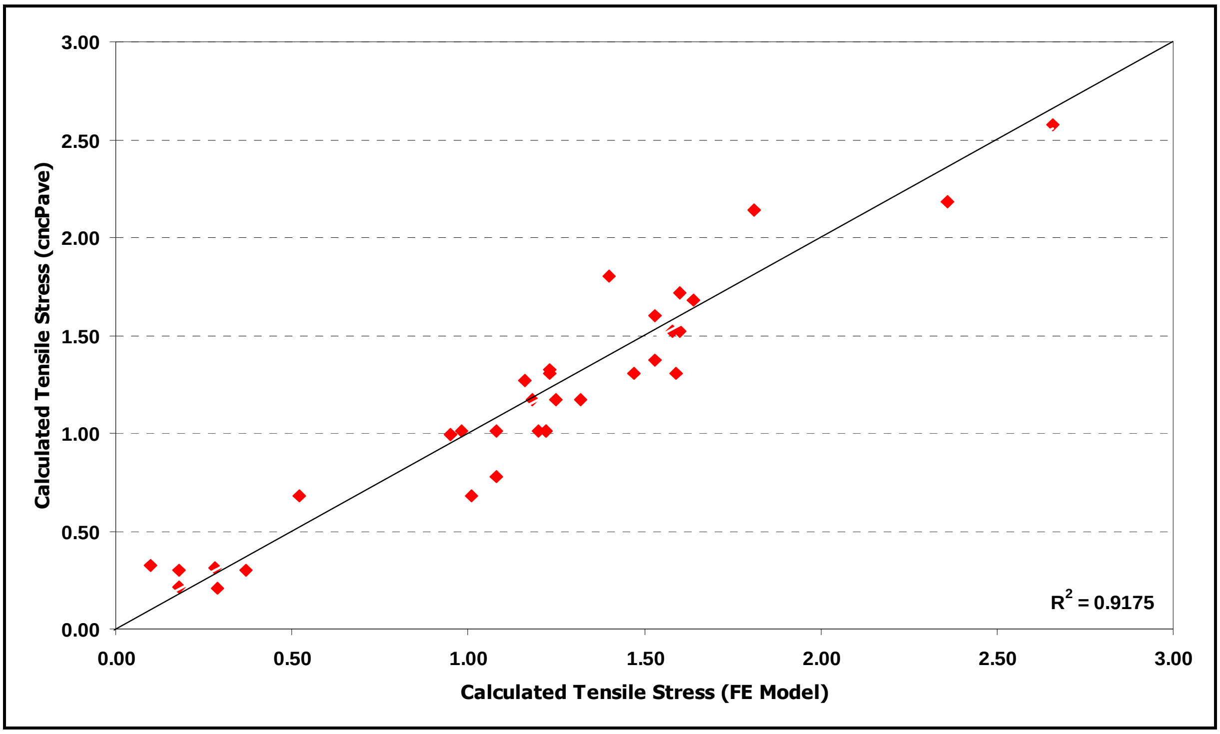



**Figure 7: FE Analysis and cncPave Calculated Tensile Stress Comparison**

### 3.1.2 HVS and FEM Mode of Failure Comparison

Combinations of FE analysis results and cncPave equations were then used to explain the development of failure as observed on the test sections under HVS testing and to establish the consequences of parameters varying during the design and construction of an UTCRCP.

The most important parameter identified is UTCRCP layer thickness as indicated in Figure 8. The maximum tensile stress on the surface due to dynamic HVS loading is shown in this figure since it is indicative of cracks that were observed during HVS testing. In calculating tensile stress, the effect of shrinkage stress was ignored since all sections were built using the same materials, the same thickness and performed under the same environmental conditions. Tensile stress in the bottom of the UTCRCP layer shows a similar trend as shown in Figure 8 when plotted against UTCRCP layer thickness.

Apart from the importance of UTCRCP layer thickness, the modelling can also be used to establish the mode of failure and to demonstrate the sensitivity of certain characteristics of UTCRCP:

1. Transverse cracks will develop as a result of shrinkage and it is inevitable that a transverse construction joint will have to be introduced into the pavement. Since steel bar reinforcement as well as some fibres will be through this joint, the joint or crack can be regarded as a hinge that allows shear but no moment to be transferred across it;
2. High stresses develop when a wheel load crosses from one side of the crack or joint to the other side. According to FE analyses, high tensile stresses develop at the bottom of the pavement when the wheel is about 450 mm from the crack. When the wheel is at the crack high tensile stresses then occur at the top of the pavement close to the tyre contact area. High compression zones develop under the wheel at the top of the crack when it crosses the crack.

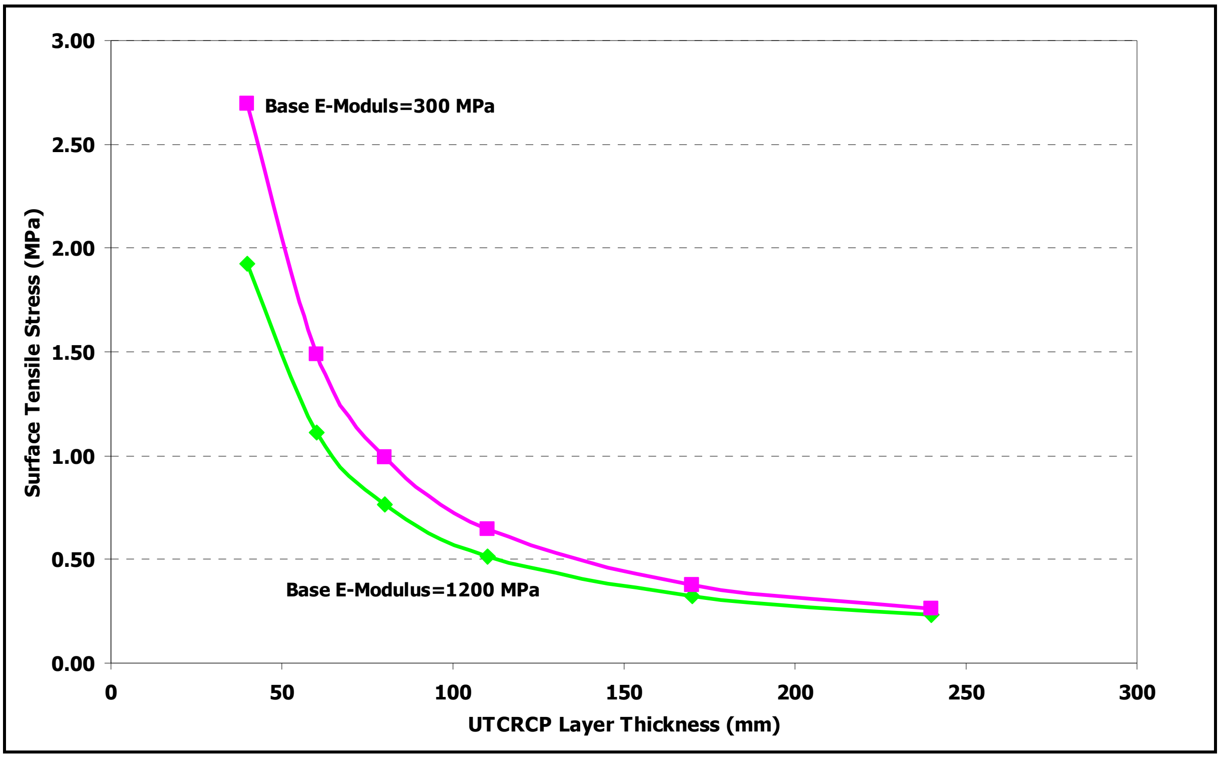



**Figure 8: Tensile Stress on Pavement Surface as a Function of UTCRCP Layer Thickness and Base Stiffness.**

The tensile stress is at its highest approximately 450mm away from the crack where a second crack will rapidly develop. The crack itself may initially not be visible, since it originates from the bottom but the stiffness of this UTCRCP layer is reduced, resulting in an increase in deflection and higher vertical stress at the top of the supporting layer. At the same time, high compression stresses develop at the top of the UTCRCP layer in the crack resulting in spalling and the risk of loss of shear resistance and water entering the UTCRCP layer. A crack will later develop from the surface down into the pavement as a result of the tensile stress on the surface;

3. Water that will enter the supporting layer through the spalled crack, results in a loss of bond and, with an increase in deflection, a void between the UTCRCP layer and the supporting layer will develop. The effect of a loss of bond is shown in Figure 11 where the maximum stress at the surface of the UTCRCP layer, calculated with cncPave, is plotted as a function of bond and crack width. Figure 12 indicates the effect of the void size once bond between the UTCRCP layer and the supporting layer is lost. The crack width in both Figures 11 and 12 are used to indicate the loss of shear and thus load transfer at the crack, and
4. Final failure appears as disintegration of the UTCRCP layer and is as a result of high tensile stress at the bottom and high compression stress at the top of the UTCRCP layer.

### 3.1.3 Correlation between HVS measured and cncPave Predicted Performance

The theoretical models that have been developed can now be used to calculate the maximum tensile stress on the surface of the pavement under HVS loading. This stress can be compared with the measured strength of the concrete mix that has been used and by applying the following equation, the number of loads to failure can be calculated:

$$N = C\,(\text{Stress/strength})^{-b} \qquad (1)$$

where the value of b has been found to be 4.5 for national highways in South Africa (Strauss et al, 2001).

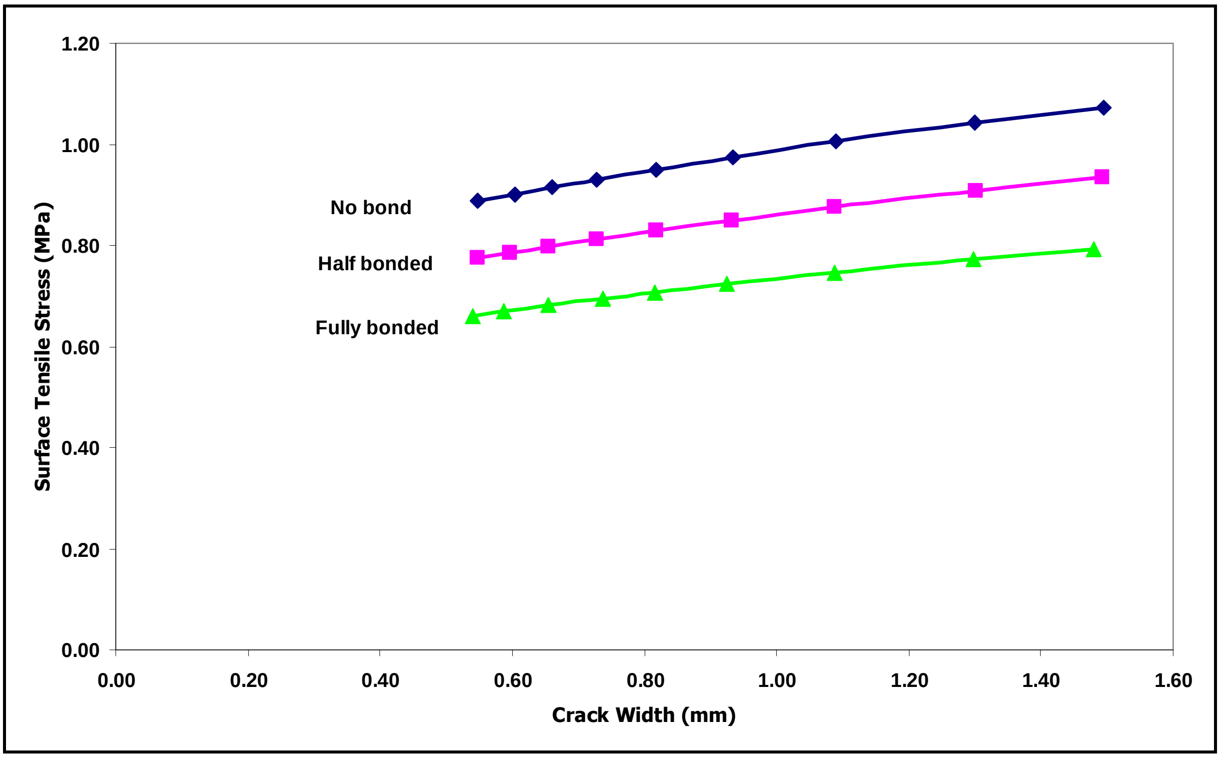



**Figure 9: Maximum Surface Tensile Stress in Relation to Crack Width and Bond Between UTCRCP layer and Support Layer.**

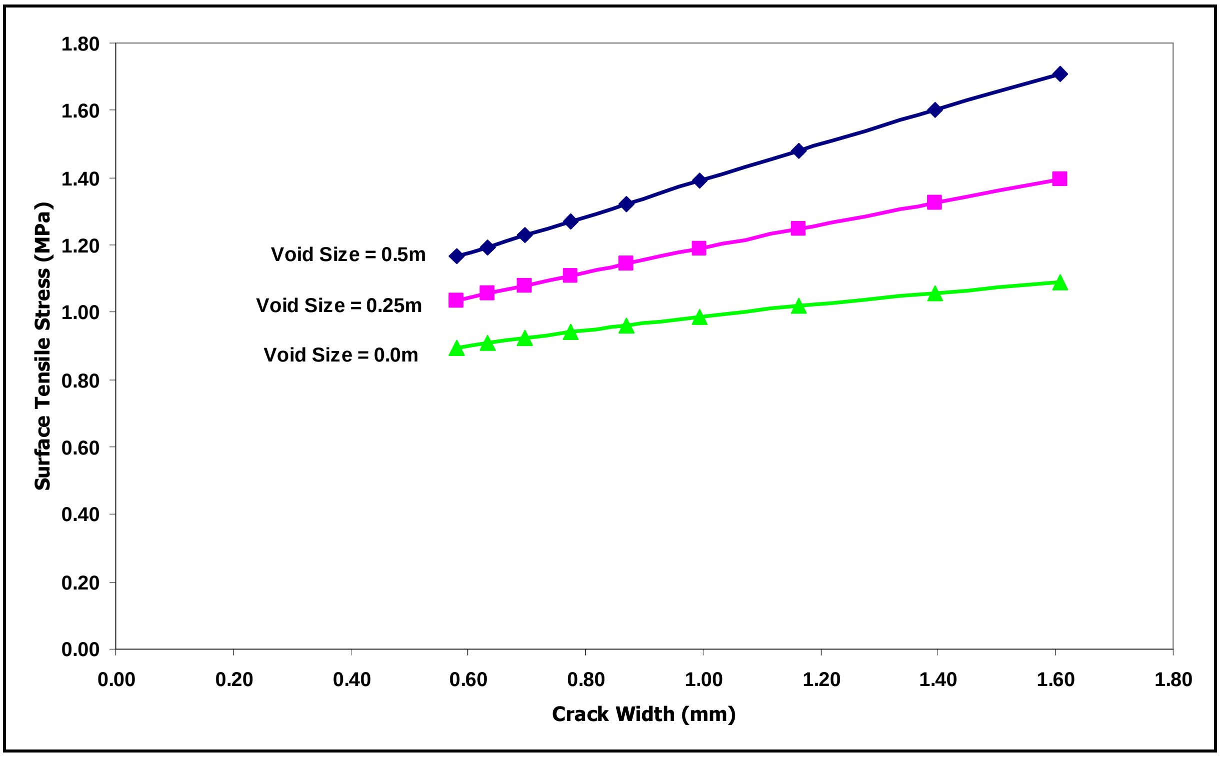



**Figure 10: Maximum Surface Tensile Stress in Relation to Crack Width and the Size of the Void between the UTCRCP Layer and the Supporting Layer.**

As indicated above, indications are that debonding between the UTCRCP layer and the base layer is initiated when water enters the pavement through cracks and/or a construction joint. This results in pumping and thus the development of an extensive void, which leads to final failure. By applying the following principles, as well as Miner's hypothesis, the performance of the different sections can then be calculated and compared with the actual values determined from HVS testing as shown in Table 6:

- The UTCRCP layer is fully bonded to the base under dry conditions;
- Whenever water is introduced on top of the pavement, debonding is initiated;
- A void develops between the UTCRCP layer and the base when the deflection on the surface has increased to about 0.6mm which normally occurs between 1.5 and 2.5 million load applications from final failure in the case of an unbound granular base and about 4.5 million load applications for a cemented base;
- Miner's hypothesis can be applied, and
- The damage coefficient is 4.5.

The results in Table 6 show some interesting trends:

- Although test 426A5 has been defined as an edge load test, the fact seems to be that loading some 300 mm from the edge resulted effectively in an interior loading condition. The predicted value in Table 6 reflects edge loading but if interior loading is considered, the predicted capacity is much closer to the actually measured value;
- When comparing tests 426A5 and 427A5 it appeared that curling did not have as big an effect on performance as would have been expected. This result is based on only one test and will have to be further investigated;

**Table 6: Tested and predicted structural performance of test sections**

| HVS Test Description | Load (wheel) | Condition | HVS Tested E80's (Million) | Predicted E80's (Million) |
|---|---|---|---|---|
| 426A5: Edge loading<br>Ave slab thickness = 45mm | Double | Dry | 2.2 | |
| | Single | Dry | 61.0 | |
| | Single | Wet | 29.0 | |
| **Total:** | | | **90.2** | **49** |
| 427A5: Interior loading<br>Ave slab thickness = 52mm | Double | Wet | 4.7 | |
| | Double | Dry | 3.0 | |
| | Double | Wet | 2.8 | |
| **Total:** | | | **10.5** | **10** |
| 428A5: Construction joints<br>Ave slab thickness = 47mm | Double | Dry | 1.7 | |
| | Double | Wet | 1.2 | |
| **Total:** | | | **2.9** | **3** |
| 429A5: Interior & chip seal.<br>Ave slab thickness = 39mm | Double | Wet | 4.3 | |
| **Total:** | | | **4.3** | **4** |
| 430A5: Interior loading<br>Ave slab thickness = 52mm | Double | Dry | 3.0 | |
| | Double | Wet | 22.7 | |
| **Total:** | | | **25.7** | **31** |
| 431A5: Interior loading<br>Ave thickness = 48mm | Double | Dry | 3.0 | |
| | Double | Wet | 15.9 | |
| **Total:** | | | **18.9** | **19** |

- The stiffness of the support did not seem to be as important as some other parameters as illustrated by the difference in life between tests 427A5 (weak substructure support), 430A5 (strong substructure support) and 431A5 (medium substructure support);
- A dry pavement gave far superior performance to a wet pavement. This is illustrated by the difference in performance between test 426A5 (edge loaded but kept dry for a long period) and test 427A5 (the same support but interior loading on a continuously wet pavement);
- The modelling of additional tensile stress due to shrinkage has not been attempted at this stage due to a lack of information on the behaviour of fibre-reinforced concrete in the field and this still needs to be done in order to increase the accuracy of the model. However, for the purpose of this study, the relative effect of this phenomenon has been assumed to be the same for all test sections because they were of similar designs and operating under the same environmental conditions;
- The most critical parameter seems to be UTCRCP layer thickness as observed by the difference in performance of sections 427A5 and 429A5, both on the same support;
- Other important parameters are bond between UTCRCP layer and base and the development of a void below the UTCRCP layer, and
- Test 430A5, showed premature failure which could be attributed to additional water since the test was performed during heavy rain storms.

## 3.2 Laboratory Testing

It is nearly impossible to accurately model the actual field behaviour of a concrete slab in a laboratory and therefore the emphasis of the laboratory investigation was on the comparative behaviour of small slabs. The slabs (750mm x 500mm) were tested in a closed loop Materials Testing System (MTS) in displacement control as shown in the photo in Figure 11. The slabs were supported to span 600 mm and loaded at third points. The load and centre point deflection were recorded at a rate of 100 Hz. As mentioned the bending test was conducted in displacement control and the deflection was increased at a constant rate of 1 mm per minute up to a value of 15 mm where after the load was released. The loads were used to calculate the bending moments and stresses in the slabs. Although the strength of these slabs can not be directly related to the strength of the UTCRCP, the ability of the slab to absorb energy is a function of the area under the load deflection graph and this energy absorption capacity would give an indication of the life expectancy of the slabs under repetitive loading. The moment resistance capacity of the slabs should never be reached in a pavement but the actual bending moments in the pavement would be a fraction of the moment of resistance and the expected life cycle of the pavement should be a function of this load fraction. The effect of changes in the composition of the pavement (such as changing the thickness, steel content or concrete strength) on the life expectancy of the pavement, should manifest itself in changes in the bending moment resistance capacity of the slabs.

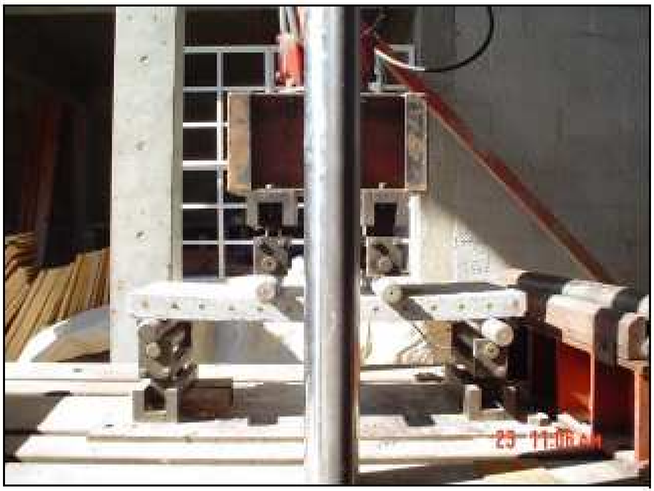


**Figure 11: MTS Test Setup**

An example of the typical results of these laboratory tests can be seen in Figure 12. These graphs are for two slabs with exactly the same composition and the repeatability of the test can be clearly seen from these results. From the graphs it can be seen that the slope of the graph changes at a deflection of approximately 1mm and a bending moment in the region of 1.6 kNm. At this point the concrete starts cracking as the flexural strength of the concrete is exceeded. At this point the stress in the beam is in the

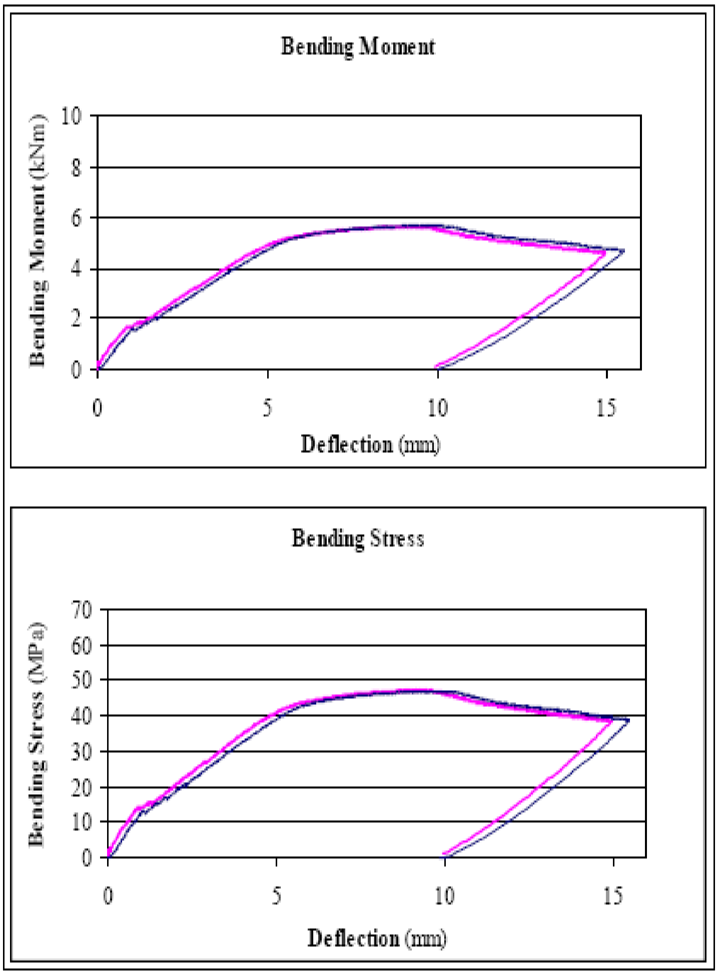



**Figure 12: Typical Laboratory Slab Result**

region of 13 MPa, which correlates well with the measured Modulus of Rupture (MOR) of the mixture. After the concrete has cracked the section stiffness is reduced, resulting in a decrease in the slope of the moment deflection graph. The stresses calculated after this point (as shown in the bottom part of the graph) are not actual stresses as the section modulus used to calculate the stresses has not been reduced to a cracked section. Although the concrete is cracked, the slab still behaves linearly elastic up to a deflection of approximately 5 mm, where the reinforcing starts yielding. As seen for the example slab, there is a residual deflection of approximately 10 mm after completion of the test. The slope of the line indicating the release of stress in the sample is similar to the slope in the elastic region of the cracked section. This indicates that the sample still had some residual strength at the end of the experiment.

Instead of using the complete graph to compare behaviour, the maximum values as well as stiffness (as indicated by the slope of the graph) and energy absorption (as indicated by the area under the bending moment graph) was used to compare different slabs. The above laboratory test procedure was used to:

- Firstly, test samples that were cut from the Phase 1 HVS test pavement were tested in the laboratory and compared to laboratory prepared samples to determine why the behaviour of the UTCRCP varied so much between wet and dry, and
- Secondly, to determine what effect different variables have on the strength of the UTCRCP. The materials in the concrete mixture used in the test section was replaced with locally available materials to determine the effect of cement, aggregate and fibre types on the properties of the UTCRCP. The thickness of the UTCRCP, concrete strength, reinforcing diameter, cover and spacing were varied to establish how sensitive the behaviour of the composite is to each of these variables.

Some of the more important results obtained from laboratory testing will be discussed in more detail in the rest of this section.

### 3.2.1 Cement Binder Type

The effect of cement type used was investigated by replacing the Contec Binder used in phase 1 testing, with South African cement, fly ash (PFA) and condensed silica fume (CSF). The effect of this can be seen in Figure 13. These results clearly indicate that it is possible to replace the propriatry Contec APS Binder (SANRA Cement) with local materials (Local Cement) and still get the same behaviour from the test slabs. Further field verification is however required.

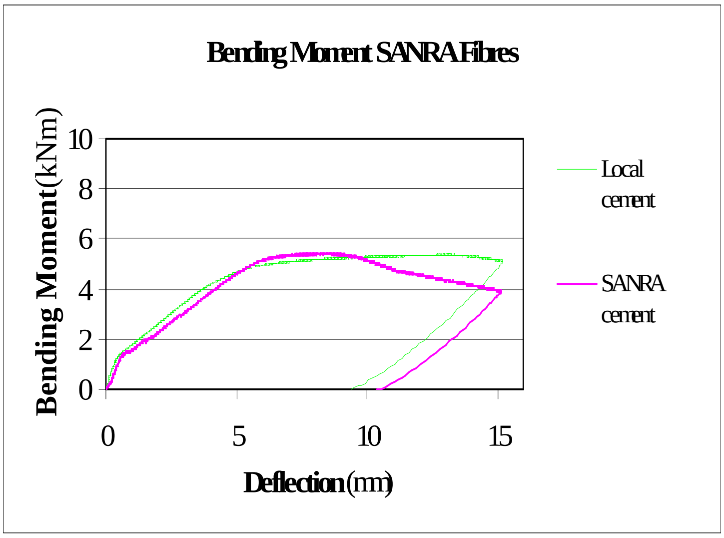
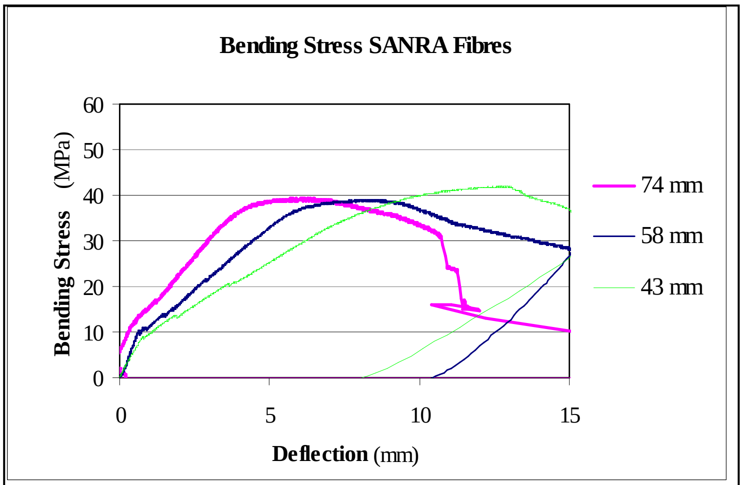



**Figure 13: Cement Binder Type**

### 3.2.2 UTCRCP Layer Thickness and Steel Fibre Type

The results obtained are illustrated in Figure 14. As seen for the original SANRAL Fibres (12mm straight steel fibre) the maximum bending stress is more or less similar for the different layer thicknesses, with the 58mm providing marginally better energy absorption (area under the curve).

For the Beakert Fibres (30mm hooked fibre) the difference is more obvious between different thicknesses, with once again the 54mm providing superior performance. When compared to the original straight fibres, the hooked fibres provide a 25% improvement in bending stress. From these results it was concluded that the optimum thickness for UTCRCP is in the range of 50 to 60mm, and that 30mm hooked fibres should be used.



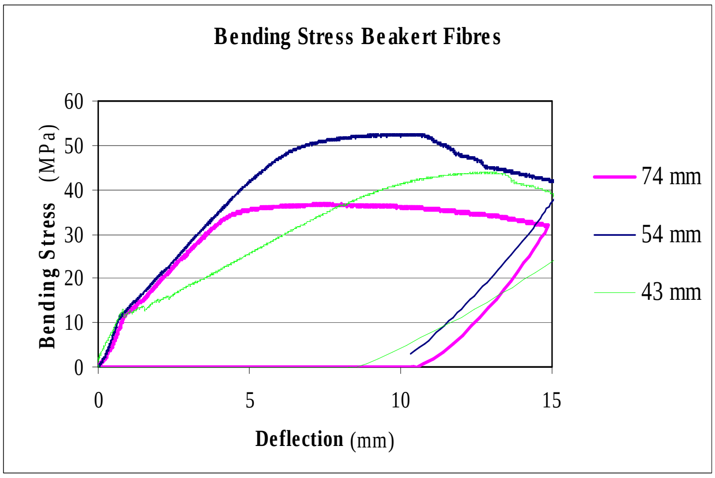




**Figure 14: UTCRCP Thickness & Steel Fibre Type**

### 3.2.3 Reinforcing Steel Content (Mesh Diameter)

The effect of reducing reinforcing steel content was investigated by using welded mesh with different diameters. The effect of this can be seen in Figure 15. These results indicate that both the maximum bending moment recorded and the energy absorbed is a function of the steel content of the slabs, with values decreasing as the steel content decreases. In an effort to balance costs, constructability and risk, the optimum seemed to be welded mesh with Ø5.6mm (Y6) mesh. Further field verification is however required.

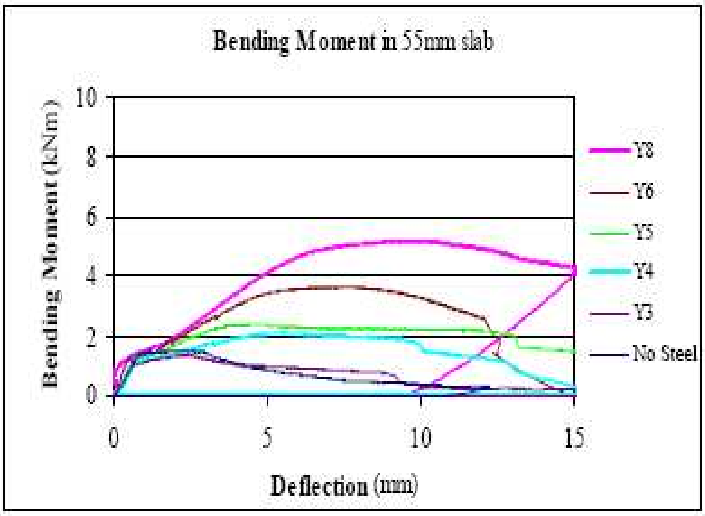



**Figure 15: Reinforcing Steel Content**

### 3.2.4 Welded Steel Mesh Size (Spacing)

The effect of increasing the steel mesh size (i.e. 50mm x 50mm to 100mm x 100mm) was investigated by using welded mesh with different mesh sizes. The effect of this can be seen in Figure 16. These results indicate that both the maximum bending moment recorded and the energy absorbed is a function of the steel mesh size within the slabs, with values decreasing as the steel mesh size increases. In an effort to balance costs, constructability and risk, the optimum seemed to be mesh size of 50mm x 50mm. Further field verification is however required.

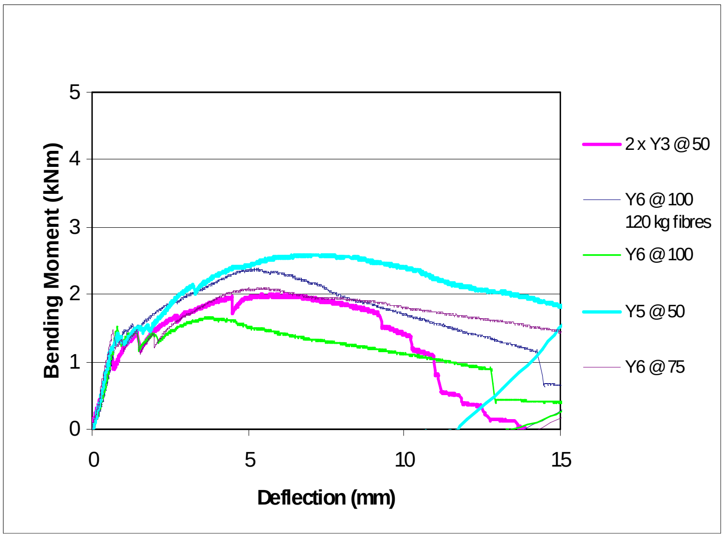



**Figure 16: Steel Mesh Size**

## 3.3 UTCRCP Test Section Reconstruction

To avoid the mixing and placement problems experienced during the construction of the phase 1 UTCRCP test sections, the decision was made for the Phase 2 test sections to mix (400 liter drum mixer) and place (double vibrating screed) the UTCRCP by using hand labour as shown in Figure 17. This enabled us to also evaluate the suitability of UTCRCP as a labour intensive construction technique. The UTCRCP was also placed directly on top of the granular base layer with no asphalt interlayer.

**Figure 17: Mixing and Placement of Phase 2 UTCRCP**

The following 8 UTCRCP test sections were constructed over the 54m x 3.7m test section by varying binder type, mesh type and steel fibre type and content:

1. UTCRCP 2.1: 50mm thick layer using Contec APS binder (used during phase 1) with Ø5.6mm steel mesh placed @ 50x50mm intervals at centre of slab and 80kg/m$^3$ of 12mm straight steel fibres;
2. UTCRCP 2.4: 50mm thick layer using Contec APS binder (used during phase 1) with Ø5.6mm steel mesh placed @ 100x100mm intervals at centre of slab and 80kg/m$^3$ of 12mm straight steel fibres;
3. UTCRCP 2.2: 50mm thick layer using RSA binder with Ø5.6mm steel mesh placed @ 50x50mm intervals at centre of slab and 80kg/m$^3$ of 30mm hooked steel fibres;

4. UTCRCP 2.5: 50mm thick layer using RSA binder with Ø5.6mm steel mesh placed @ 100x100mm intervals at centre of slab and 80kg/m$^3$ of 30mm hooked steel fibres;
5. UTCRCP 2.3: 50mm thick layer using RSA binder with Ø5.6mm steel mesh placed @ 50x100mm intervals at centre of slab and 100kg/m$^3$ of 30mm hooked steel fibres;
6. UTCRCP 2.6: 50mm thick layer using RSA binder with Ø5.6mm steel mesh placed @ 100x100mm intervals at centre of slab and 100kg/m$^3$ of 30mm hooked steel fibres;
7. UTCRCP 2.4: 50mm thick layer using RSA binder with Ø4.0mm steel mesh placed @ 50x50mm intervals at centre of slab and 80kg/m$^3$ of 30mm hooked steel fibres;
8. UTCRCP 2.8: 50mm thick layer using RSA binder with 2xØ4.0mm steel mesh placed @ 50x50mm intervals at centre of slab and 80kg/m$^3$ of 30mm hooked steel fibres;

## 3.4 UTCRCP APT Results

### 3.4.1 Load Repetitions

The main objective of phase 2 Accelerated Pavement Testing (APT) testing was to evaluate the potential of the improved UTCRCP under the Heavy Vehicle Simulator (HVS). For Phase 2, the following standard HVS test regime was adopted for all tests:

- Test Wheel Load = 80kN on dual truck tyres (equivalent to 16 ton axle);
- Test Tyre Pressure = 800 kPa;
- Load Application = Canalized bi-directional;
- Load Speed = 9 km/h, and
- Surface Water: Cycle of 120,000 dry repetitions followed by 40,000 wet repetitions throughout the test.

To date a total of 3 HVS tests have been completed with one still ongoing on the phase 2 test sections constructed at Heidelberg Traffic Control Centre (HTCC), the results of which are summarised in Table 7. Table 8 give some details of the parameters being varied as well as details of the HVS testing sequence for phase 2. In general the phase 2 test sections have shown improved performance compared to phase 1, despite containing 50 % less reinforcing steel.

**Table 7: Phase 2 – HVS Load Regime**

| Test Wheel Load (kN) | HVS Repetitions Per Test | | | |
|---|---|---|---|---|
| | **432A5** | **433A5** | **434A5** | **435A5** |
| 40 | 40,000 | 40,000 | 40,000 | 40,000 |
| 60 | 40,000 | 40,000 | 40,000 | 40,000 |
| 80 | 960,000 | 840,000 | 560,786 | 1,320,000 |
| 80 | 315,592 | 256,538 | 170,000 | 410,000 |
| **Total Reps** | **1,355,592** | **1,176,538** | **810,786** | **1,810,000** |
| **Total E80's (n=4.5)** | **29,151,363** | **25,099,833** | **16,823,810** | **39,433,442** |

Notes:

1...\ The total amount of repetitions in the table indicates the amount of HVS repetitions (Please Note: E80 = $(Load/40)^{4.5}$ since it is wheel load and not axle load) to failure.
2...\ 80kN wheel loads was applied in repetitive cycle of 120,000 dry followed by 40,000 wet (Shaded Area).
3...\ Test 434A5 reflects the total applied to the test section, the section did have initial failure after 259 000 repetitions (due to subsurface drainage problems), after which the HVS test section was slightly moved.
4...\ Test 435A5 is still in progress.

**Table 8: Phase 2 - HVS Test Summary**

| HVS Test No | Test Description |
|---|---|
| HVS 432A5   | To establish performance 50mm thick UTCRCP layer using Contec APS binder (used during phase 1) with Ø5.6mm steel mesh placed @ 50x50mm intervals at centre of slab and 80kg/m3 of 12mm straight steel fibres |
| HVS 433A5   | To establish performance 50mm thick UTCRCP layer using RSA binder with Ø4.0mm steel mesh placed @ 50x50mm intervals at centre of slab and 80kg/m3 of 30mm hooked steel fibres |
| HVS 434A5   | To establish performance 50mm thick UTCRCP layer using RSA binder with Ø5.6mm steel mesh placed @ 100x100mm intervals at centre of slab and 80kg/m3 of 30mm hooked steel fibres |
| HVS 435A5  | To establish performance 50mm thick UTCRCP layer using RSA binder with Ø5.6mm steel mesh placed @ 50x100mm intervals at centre of slab and 100kg/m3 of 30mm hooked steel fibres. Still in Progress. |

### 3.4.2 Surface Deflection

The 40kN surface deflections against the number of repetitions of all the tests can be seen in Figure 18. The figure displays the maximum deflection per test as captured by the JDMDs. The impact of the dry and wet cycles (shaded areas) is evident, and as seen all test sections failed during a wet cycle. Also interesting to note is the decrease in deflection that occurred on test 432A5 during wet cycles, it is postulated that water filled the void and provided additional support.

When comparing test 432A5 (Ø5.6mm) and 433A5 (Ø4.0mm) it is evident that once deflection exceeded the critical threshold the section with thinner steel failed faster and at much lower maximum deflection. It is postulated that this is due to the fact that area of steel of Ø4.0mm steel mesh is only 50% of that of Ø5.6mm steel mesh.

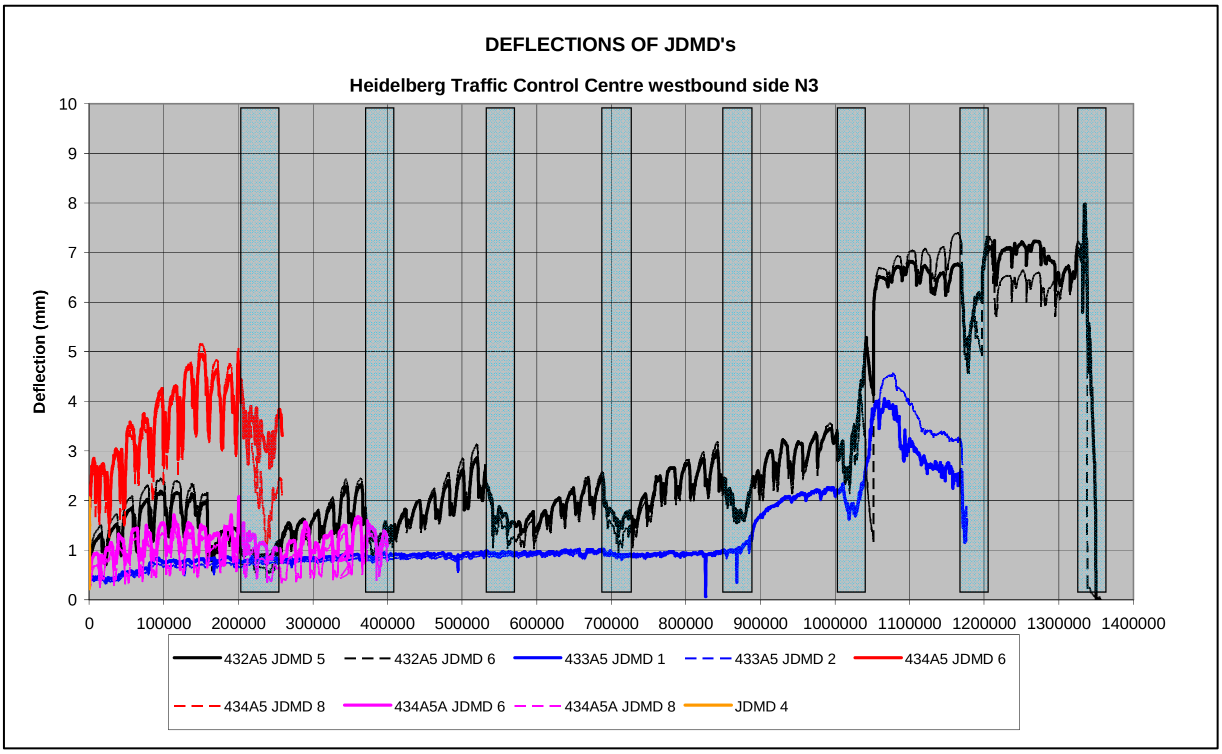



**Figure 18: Peak 40kN Surface Deflection**

### 3.4.3 Surface Permanent Deformation

The permanent deformations as recorded by the same instruments as above are shown in Figure 19. The same conclusion is drawn regarding the permanent deformation of the pavement sections. As with the phase 1, it is once again notice that as soon as the permanent deformation exceeds ± 5mm failure follow soon thereafter.

### 3.4.4 Visual Deterioration

This mechanism of failure is similar to what was observed during phase 1, although generally after a higher number of repetitions now. Table 9 shows the typical crack patterns observed at the failed area for the various tests. The noticeable difference is the fact that sections with hooked steel fibres (433A5, 434A5, 435A5) no longer display the "explosive" failure where concrete at surface debonds from the reinforcing mesh.

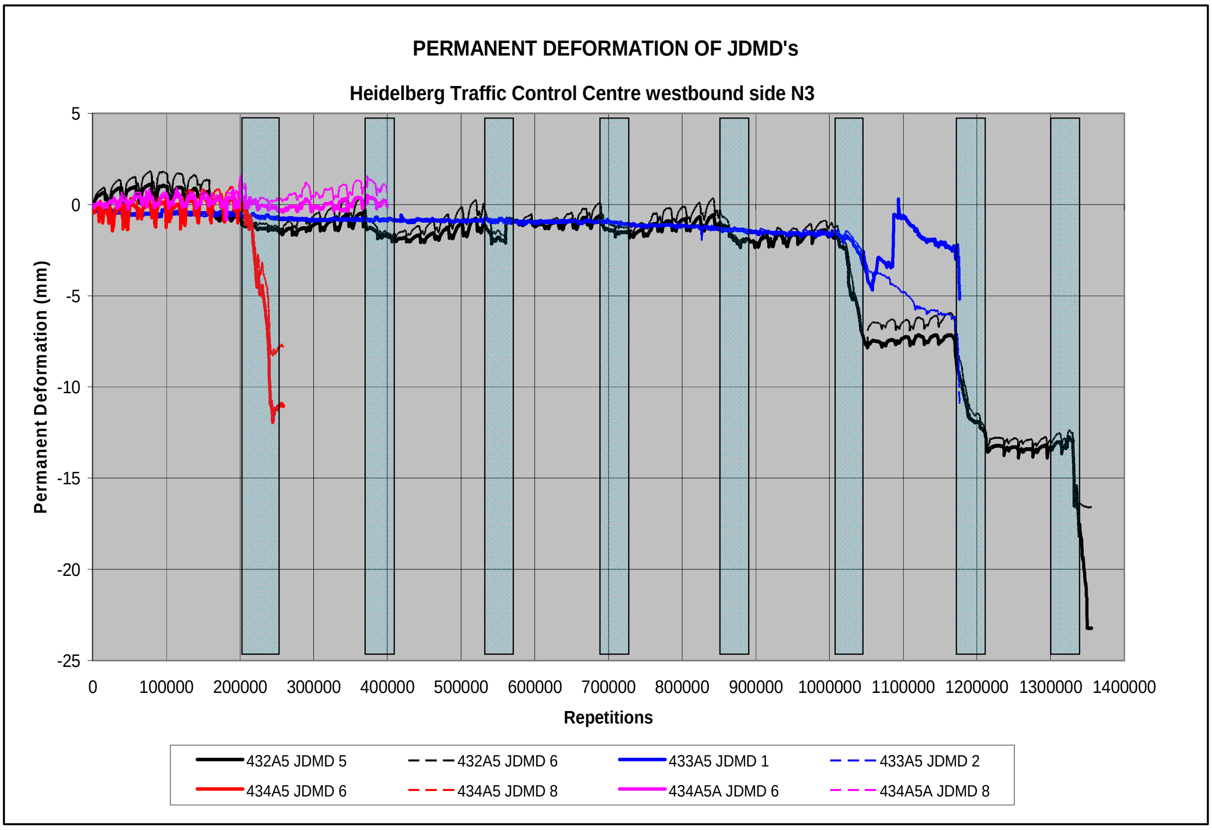
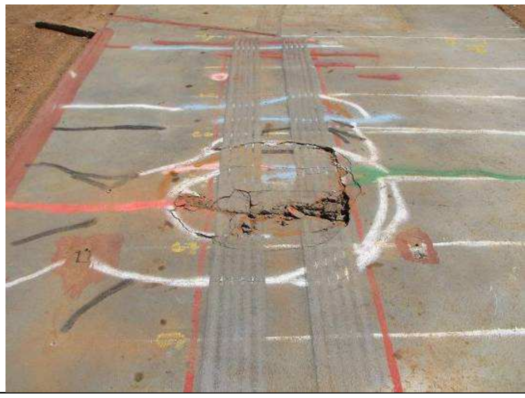
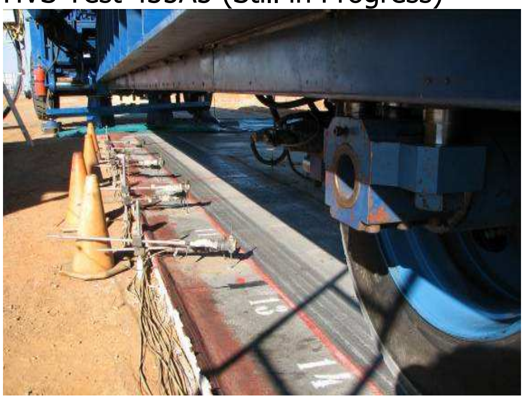



**Figure 19: Surface Permanent Deformation**

**Table 9: Phase 2 - HVS Test Section Failure Photo's**

| | |
|---|---|
| HVS Test 432A5  | HVS Test 433A5  |
| HVS Test 434A5   | HVS Test 435A5 (Still in Progress)  |

## 3.5 HVS Phase 2 Field Test Conclusions

At this stage only the following preliminary conclusions can be made from the Phase 2 tests:

1. Excluding the original phase 1 dry test (426A5), the phase 2 test sections have shown a general improved performance despite containing less than 50 % of the original reinforcing steel;
2. The phase 1 mixing and placement problems seemed to have been adequately addressed, and no durability issues related to hand placement of UTCRCP can be identified at this stage;
3. The use of the hooked steel fibres have prevented the "explosive" failures observed during phase 1;
4. The UTCRCP layer only fails in vicinity of a crack that is wide enough to allow surface water to penetrate, then erode the base material to create a void;
5. Excluding test 434A5 (sub surface drainage problem) the minimum expected life for the UTCRCP on a base with 0.6mm surface deflection (40kN) are 25 million E80's. It is believed that this is a conservative estimate due to the load regime of the HVS tests.

# 4. CONCLUSIONS

The purpose of the study was to test the suitability of a UTCRCP overlay on a flexible pavement near the end of its life. Test sections were constructed using a different combination of support stiffnesses, construction joints in the UTCRCP layer, and changing the environment by adding water. Subsequently FE analyses were used to model the behaviour of the pavement and to simulate the development of distress. The model was then used to determine the relative sensitivity of design parameters and to further develop the design program cncPave for use by practitioners. The following can be concluded from the study:

- FE modelling closely simulated observed behaviour under HVS loading;
- Although the FE modelling indicated high tensile stress at the bottom of the UTCRCP layer, about 450 mm from the joint or crack, cracking at this position of high tensile stress was not as prominent as circular surface cracks and thus tensile stress at the surface was used to simulate observed distress;
- The thickness of the UTCRCP layer as well as the presence of a joint or crack is critical to the performance of a UTCRCP;
- The relative position of the longitudinal steel reinforcement is not important but placing it closer to the top of the UTCRCP layer reduces compressive stress in a crack or joint thereby reducing the risk of spalling and the access of surface water;
- Debonding between the UTCRCP layer and the support directly below the UTCRCP layer occurs, probably due to water entering the pavement, and this leads to an increase in stress;
- The development of a void as a result of debonding, water entering and an increase in deflection is detrimental to the performance of a UTCRCP;
- Curling did not seem to have a significant effect on the performance of the UTCRCP layer,
- For the test conditions the UTCRCP provides minimum life of 25 million E80's.

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