ROODEKRANS THIN CONCRETE EXPERIMENT SECTIONS 4, 5 AND 6: CONTINUOUSLY REINFORCED THIN CONCRETE PAVEMENTS

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ABSTRACT

A joint experiment on the performance of thin concrete pavements was conducted at Roodekrans, Gauteng for the past 2 years. The decision to include three sections of thin Continuously Reinforced Concrete Pavement (CRCP) in the Roodekrans experiment was the outcome of the observations of one of the authors during visits to Iowa during the 1970s and 1999, where the good performance of a 4" (100 mm) CRCP was observed. Based on the information collected during these visits, it was decided to construct a length of 40 metres of CRCP with concrete thicknesses of 50 mm, 75 mm and 100 mm (Sections 4, 5 and 6 of the experiment) reinforced with a 200 x 200 x 5.6 mm welded wire mesh. This paper reports on some of the practicalities and performance of the three sections of CRCP constructed as part of the Roodekrans experiment.

1. BACKGROUND AND INTRODUCTION

In 1979 one of the authors attended the Low Volume Roads Conference in Iowa, USA. A thin concrete road (100 mm thick), both un-reinforced and reinforced with a *6" x 6" x $^{1}/_{8}$"* welded steel mesh was shown to attendees to the conference. The performance of the reinforced pavement after 15 years of traffic was most impressive (especially while carrying 1 100 vehicles per day with 4 to 4.5 per cent heavy vehicles). During a second visit to Iowa (1999) only a report on the work and the findings recorded could be found as the road was reconstructed along with other experimental sections that were in poor condition (Iowa Highway Research Board, 1989). Based on the observations in Iowa, the following three issues were identified:

- Joint failures appeared to be a major issue spot on concrete roads;
- Quarter Point failures due to shaping round gravel road into two flat sections, resulting in the fill from the cut-to-fill not being adequately compacted, caused concrete failure, and
- Mud Spot failure, resulting from inadequate support due to poor drainage / standing water on high PI support material, resulted in concrete failure.

These observations led to the formulation of the following guidelines for thin reinforced concrete roads:

- If detailed attention was given to the support layers, and the drainage and ingress of water into these layers were controlled, Quarter Point and Mud Spot failures could be reduced.
- If the concrete were laid continuously without joints using limited steel mesh, joint failures will be prevented due to:

    - Limited ingress of water at the joints or possible cracks;
    - No pumping;
    - Possible improved spreading of the wheel loads, and
    - Thinner slabs and more flexible slabs.

Based on these observations the three experimental sections were added to the Roodekrans experiment. This paper focuses on some design and material issues, performance and guidelines for these thin CRCP sections.

2. THE ROODEKRANS EXPERIMENT: SECTIONS 4, 5 AND 6

2.1 Philosophy

The basic design philosophy for the thin CRCP sections can be summarised as follows:

- If it were possible to construct a subbase with a very low flexibility, a thin concrete pavement could be constructed over this foundation, and the thin concrete would not develop fatigue failure;
- If the concrete were laid continuously without joints, cracking could only occur due to shrinkage (due to curing of the concrete) or reduction of overnight temperatures, and
- If steel mesh were introduced and if the concrete were laid on a warm day, this would probably overcome both joint problems and shrinkage problems due to curing or temperature differentials.

At the time of construction, the interest in the thin concrete was essentially related to the upgrading of low volume roads in townships, as the construction required little major plant and was labour friendly.

Construction of the concrete surface was commenced and completed during the first week of February 2002. The road concrete surface was allowed to cure for four weeks and was opened to traffic in early March 2002.

2.2 Design issues

Before construction started, Dynamic Cone Penetrometer (DCP) tests as well as CBR and indicator tests were carried out on the existing road and its materials. Using this information surface deflection values were estimated. In order to reduce the deflections to supply a low-deflection support to the CRCP layer, a 125 mm stabilised subbase was added to the structure.

Three experimental sections were designed and constructed, viz. 100 mm, 75 mm and 50 mm thick concrete respectively, with 200 mm x 200 mm x 5.6 mm mesh reinforcing placed in the middle of each section. The 50 mm section was included mainly to establish the lower limit of thickness that could be considered.

2.3 Subgrade and Subbase materials

The material for the subbase consisted of weathered Halfway House granite with a classification that varied between A1 - a (o) and A2 - 4(o) (G5 and G6).

Average CBR at 100% AASHTO	=	75
Average PI	=	6
Average GM	=	2.50

The existing subgrade (150 mm) was compacted to 99.1 per cent of Modified AASHTO density, with an average density of 2 143 kg/m^3. The subbase was stabilised with 2 per cent Duratech and compacted to 100 per cent Modified AASHTO density. The average Unconfined Compressive Strength (UCS) of the stabilised material was 1.95 MPa and the material was Non Plastic.

In order to accommodate the different thickness of concrete on uniform subbase levels, 50 mm and 25 mm of emulsion treated material was placed on top of the cement stabilised subbase on the sections where the 50 mm and 75 mm concrete sections were to be constructed. This was done in order to use 100 mm shuttering for the total length of the three sections. The levels of the three subbase sections were checked by means of string lining between the shutters.

The 40 m of roadway (3.6 m wide) was cast continuously without any joints (Table 1).

2.4 Concrete

The same concrete was used on all the experimental sections at Roodekrans to facilitate the construction process. This meant that a concrete with a maximum aggregate size of 19 mm and a 28 day strength of 30 MPa (COLTO specification) was used. Normally, a finer aggregate would have been specified for the CRCP sections. As the CRCP sections were cast continuously, anchor end blocks were constructed between Sections 3 and 4 and between Sections 6 and 7 (approximately 40 m apart). The average strength of the concrete used for the CRCP was 36 MPa.

2.5 Reinforcing Mesh

A 200 mm x 200 mm x 5.6 mm welded wire mesh was used and placed on stools at the centre of the pavement slab at the spacing as indicated in Figure 1. The percentage of steel in each section is shown in Table 2. The steel was placed at the middle of each slab thickness. Normally, 0.25 per cent temperature steel is specified for water retaining structures and 0.6 per cent for CRCP. This compares with the between 0.12 per cent and 0.25 per cent steel used for the CRCP. The ratio of steel to concrete for the 50 mm section would appear better balanced than for the 75 mm and 100 mm sections.

Table 1: Site Layout of Sections 4, 5 and 6.

Chainage	Section	Length	Road structure
Ch 40.0 Ch 53.5	Section 4	13.5m	50 mm CRCP (200x200x6mm mesh) 50 mm ETB 125 mm Stabilised gravel subbase 150 mm in situ compacted subgrade gravel
Ch 67.0	Section 5	13.5m	75 mm CRCP (200x200x6mm mesh) 25 mm ETB 125 mm Stabilised gravel subbase 150 mm in situ compacted subgrade gravel
Ch.80.0	Section 6	13.0m	100 mm CRCP (200x200x6mm mesh) 125 mm Stabilised gravel subbase 150 mm in situ compacted subgrade gravel

Table 2: Percentage steel in Sections 4, 5, and 6.

Section	Thickness of concrete	Percentage steel
4	50 mm	0.25
5	75 mm	0.16
6	100 mm	0.12

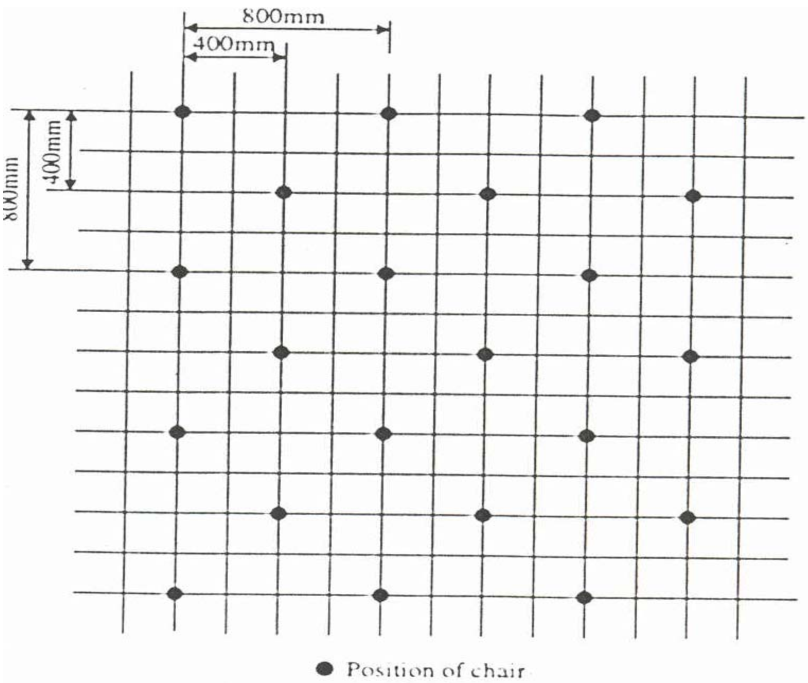

Figure 1: Spacing of chairs/stools to support the reinforcing mesh.

3. TRAFFIC

In order to quantify failure of low-volume concrete roads within a short space of time, all the experimental sections were designed to carry between 40 000 and 60 000 equivalent 80 kN axle loads. Actual traffic consisted of only heavy vehicles, with an average of 4 E80s per vehicle and 1.4 E80s per axle. The traffic count up to November 2004 (date of preparation of this paper) stood at approximately 395 000 E80s (Table 3), of which 35 per cent were overloaded. The section is still performing well and is carrying approximately 3 000 heavy vehicles per month (equivalent to approximately 12 000 E80s/month OR 400 E80s/day).

As the 3.6 m wide pavement did not have shoulders or an adjacent traffic lane, the applied traffic loads were channelised. A factor of 1.5 could conservatively be applied to estimate the equivalent traffic, should a concrete pavement with two lanes and shoulders be considered. This means that the experimental sections would have effectively carried approximately 600 000 E80s to date and are still performing well (i.e. no maintenance would have been required for a period of 40 years for a road carrying approximately 40 E80s/day).

Table 3: Record of Traffic.

Period	Heavy Vehicles	Axles	E80s (Approximate)
12/03/02 – 5/08/02	16 716	49 799	69 500
12/03/02 – 30/11/03	66 092	196 902	275 000
12/03/02 – 30/11/04	96 116	299 321	395 000

4. PAVEMENT PERFORMANCE

4.1 Deflection Calculation

It was considered appropriate at an early stage of planning to determine an indication of the expected deflections of the road, based on information available from the in situ materials. Using an empirical equation (Equation 1) developed from DCP investigations, estimates of the resilient moduli (E-values) for the various layers were calculated (De Beer, 1991).

$$E=10^{(3,04758 - 1,061666 \times \log10 (DN))} \qquad \text{Equation 1}$$

Next, the elastic deflections for each of the layers were calculated using Equation 2 and assumptions of a 40 kN wheel load and a 300 mm diameter contact area.

$$dl = \frac{P/A}{E} \times l \qquad \text{Equation 2}$$

Where

dl	=	deflection
l	=	depth of layer in question
A	=	area determined by assuming the stress is dissipated with depth and assuming 1:1½ vertical to horizontal ratio of the cone formed by the 300 mm diameter contact area at the surface
P	=	Applied load

The sum of the individual layers' deflections provides an indication of the total deflection for the road.

4.2 Average Deflections for Sections 4, 5 and 6

The deflections were also measured using the Falling Weight Deflectometer (FWD) after construction, and these measured deflections as well as those calculated are shown in Tables 4 to 6. It can be seen that the comparison between the calculated and measured deflections improves as the whole pavement structure is incorporated (Table 6 versus Tables 4 and 5) – this is probably due to the fact that Equation 1 was originally developed using data for complete pavement structures.

A total of 70 000 E80s was applied to the experimental sections between the 11/03/02 and the 5/08/02 deflection measurements shown in Table 6.

Table 4: Subgrade deflections.

Section	4	5	6	Average
FWD deflection [micron]	1 426	1 332	1 519	1 384
Predicted from DCP [micron]	930	1050	975	985

Table 5: Subgrade + Stabilised Subbase.

Section	4	5	6	Average
FWD deflection [micron]	808	894	867	856
Predicted from DCP [micron]	648	722	637	669

Table 6: Subgrade + (150 mm) Stabilised Subbase + Concrete.

Section	4	5	6	Average
FWD (11/03/02)	428	503	227	386
FWD (27/05/02)	440	446	322	402
FWD (05/08/02)	513	518	370	467
Predicted from DCP	590	573	552	572

4.3 General section performance

The performance of the pavement is mainly judged visually. There was evidence of cracking on the pavement prior to opening to traffic (Sections 4 and 5 - Photo 1). This is perceived to be due to poor workmanship insofar the thickness and uniformity of thickness was not maintained during construction. Further, the steel mesh was not placed at the middle of the sections, resulting in a blow-up that occurred at the transition between Sections 4 and 5 (Photo 2) in November 2003 after carrying some 275 000 E80s. The blow-out was repaired and is performing well since.

Initially it was observed that Section 4 performed better than Sections 5 and 6. With time, more cracks appeared on all three sections (Photos 3 and 4). However, the road is currently still performing well under the indicated traffic load.

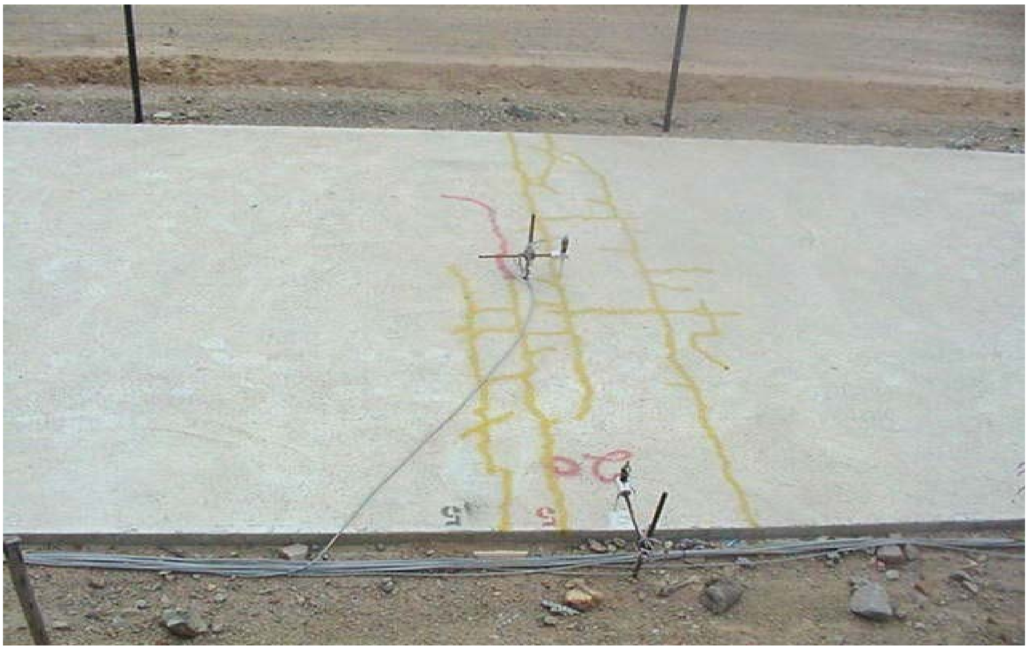

Photo 1: Shrinkage cracks at transition between Sections 4 and 5.

Photo 2: Blow-up in November 2003.

Photo 3: Development of cracks as at 20/01/2005 after 400 000 E80s (View of least cracked portion on Section 4 (50 mm)).

Photo 4: Development of cracks as at 20/01/2005 after 400 000 E80s (View of worst cracked portion Section 4 (50 mm)).

4.4 Curling of Thin Concrete Slabs

Curling of thin concrete slabs was considered a cause for concern at the inception of the project, especially as the narrow pavement and tracking of the heavy trucks would subject the pavement to severe test loading. It was anticipated that if curling did result, there would be initial cracking along the centre line of the slabs. No evidence of centre line longitudinal cracking was observed to date.

4.5 Temperature Stresses

After the blow-out that occurred in November 2003, it was decided to determine the kind of temperature stresses that could be generated in the system. The following assumptions were made in this exercise:

- The range of ambient temperatures for the region was between - 4°C and 55°C, and
- The temperature during the laying and curing of the concrete was between 20°C and 25°C.

It was found that the stresses generated by the expected range of minimum and maximum temperatures were well within the working stresses of concrete and steel. The temperature stresses for a 40 meter length of pavement for a range of temperature differentials from 10°C to 40°C was calculated using Equations 3 and 4. The E-value for a 30 MPa concrete was assumed as 3×10^4 MPa, while the Co-efficient of expansion (Cc) for concrete was assumed to be between 12×10^{-6} and 14×10^{-6} (Fulton's Concrete Technology, 1986). The calculated stress and expansion values are shown in Table 7. These values will be influenced by bonding and / or friction between the concrete and the subbase.

$$\underline{P/A} = E \cdot dl/l \qquad \text{Equation 3}$$

$$dl/l = Cc \times dT \qquad \text{Equation 4}$$

Table 6: Stresses due to temperature changes.

dT (°C)	*dl/l*	Stress (P/A) (MPa)	Expansion *dl* (mm)
10	0,00012	3,60	4,8
20	0,00024	7,20	9,6
30	0,00036	10,80	14,4
40	0,00048	14,40	19,2

5. CONCLUSIONS

The following conclusions are drawn from the experiences with these three experimental sections:

- If the deflections of the foundation below a thin CRCP can be reduced to and contained at 0.65 mm, it would be possible to construct a CRCP (30 MPa concrete strength, 50 mm thick) using a 200 mm x 200 mm x 5.6 mm mesh, for a traffic loading of 400 000 E80s and obtain a serviceable pavement;
- The expected curling of the concrete has not resulted in longitudinal cracking on the centre line of the pavement in this experiment;
- The method of laying the concrete is labour friendly and can be easily developed for small contractors constructing township streets;
- It was possible to efficiently repair the blow-out of the concrete, and this repair has taken approximately 125 000 E80s since November 2003 and is still in good condition, and
- It would be possible to construct these CRCP pavements successfully for low and medium volume roads, eliminating the problems associated with joints, and reducing the cost of road maintenance.

6. GENERAL COMMENTS

6.1 Mthatha Quarry Access Road

Arising out of the satisfactory performance of Section 4 (50 mm concrete) it was recently decided (in collaboration with the Department of Roads and Public Works of the Eastern Cape) to proceed with the construction of 2.9 km of road serving a quarry and a local community near Mthatha with a 50 mm CRCP on a 100 mm ETB. The ETB and concrete surface, which are suited to construction by labour intensive methods, are to be constructed by labour using light plant in keeping with the objectives of the Expanded Public Works Programme.

6.2 Way forward

Based on the experience on the Roodekrans project, it is known that the thin CRCP can perform well, although the theoretical basis is not yet clearly established. CSIR is at present working with the University of Pretoria to assist in establishing the theory behind the performance of thin concrete pavements.

Serious consideration should be given to the use of thin CRCP overlays on rehabilitation projects where the deflections of the existing pavements are appropriate for anticipated traffic loading.

7. ACKNOWLEDGMENTS

The team would like to acknowledge the assistance and interest of Dr Pieter Strauss and Mr Bryan Perrie of C&CI in this project. The permission of the Director of CSIR Transportek for publishing this paper is also acknowledged.

8. REFERENCES

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