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RESEARCH REPORT 381

CONCRETE PAVEMENT TRIALS IN ZIMBABWE

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Main subject area: Concrete Pavement Research
Theme: Urbanisation/Transport
Project title: PC Concrete Pavements
Project reference: ODA/3.4

This document is an output from an ODA-funded research project, carried out for the benefit of developing countries.

Overseas Centre, TRL, 1993
ISSN 0266 5247

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CONCRETE PAVEMENT TRIALS IN ZIMBABWE

ABSTRACT

In 1985 the Overseas Unit of the Transport Research Laboratory assisted the Ministry of Transport in Zimbabwe to design and construct four trial sections of concrete road pavement with a total length of 1.42 kilometres. The objective was to demonstrate the viability of concrete pavements in a region where they are not used and to generate enough data to prepare suitable specifications for the conditions common in the drier regions of Africa. This report describes the designs, summarises the condition surveys that have taken place since the road was opened to traffic and draws initial conclusions concerning design and material specifications.

The average annual rainfall for the construction site is 670mm whilst the traffic loading on the trial sections is in the order of 50,000 equivalent standard axles (ESAs) per year.

This report provides a description of the test sections and their performance over the first six years showing roughness and skidding resistance to be almost unchanged while deflections and joint condition have deteriorated. The effect of slab warping is also found to be significant under these climatic conditions.

Overseas Unit Working Paper 210 contains a detailed description of the construction of these pavements; OU WP 236, 261, 264 and PR/OSC/025/93 contain details of subsequent condition surveys, which are summarised and discussed in this report.

1. INTRODUCTION

Most countries in Africa have very little experience of PC concrete road pavements. By tradition, paved roads have been surfaced with asphaltic concrete or bituminous surface dressings. However, over the past decade several of these countries have shown interest in the PC concrete alternative. The reasons for this are given as:

a) Few African countries have indigenous oil stocks and so pay for bitumen with scarce foreign exchange which must be earned by exporting primary commodities. Many of them have limestone and coal with which to make cement and most have some cement making capability.

b) It is preferable, strategically, to produce road binders within a country rather than depend upon other sources outside the control of government.

c) Concrete pavements require less maintenance than bituminous pavements. If true, this is an important advantage, especially where roads are failing prematurely for lack of routine and periodic maintenance (World Bank, 1988).

Because of this growing interest in concrete as a paving material, four trial sections were constructed on the Gweru-Mvuma road in Zimbabwe in 1985 as a joint venture between the Zimbabwe Ministry of Transport (MOT) and the UK Transport Research Laboratory (TRL). The objectives of the collaborative project were:

- to demonstrate the viability of concrete pavements as a low maintenance alternative and

- to develop the most appropriate designs for the climate and traffic in Zimbabwe, which is typical of large areas in Africa.

2. PAVEMENT DESIGN

In order to predict the traffic load that the trial sections would sustain during a nominal 20 year life, a 24 hour, 7 day axle load survey was carried out in February 1985. This gave a two-way total for the week of 570 ESAs, using the equation from Rolt (1981)

$$\text{ESA (for each axle)} = \left(\frac{\text{axle load in kg}}{8{,}160}\right)^{4.5}$$

Growth rate was difficult to predict because the site for the trial sections formed part of a road which was likely to generate traffic on completion in 1989 when the final part was due to be upgraded from a single lane to a two-lane carriageway. No growth would result in approximately 0.6 million ESAs over 20 years. It was estimated that the improvements would probably bring this total to 1 million. The axle load spectrum, as measured in 1985 and during subsequent surveys, is shown in the Appendix in Table A1 with damaging factors calculated from the axle loads. Table A2 contains traffic counts made by the local District Office.

For the relatively low design life of one million ESAs, three internationally used design methods for determining slab thickness (CPCA, 1984; AASHTO, 1975; RRL, 1970) gave the results shown in Table 1 for three different sub-base strengths, chosen as typical for the local materials.

For the Zimbabwe trials, four designs were prepared using three different thicknesses of concrete slab, two sub-bases and different joint designs, with the purposes of determining the most economical slab/sub-base combination, and the most suitable joint design for the materials, climate and traffic. Figure 1 shows the vertical

TABLE 1

Slab design thickness

CBR of Sub-base	TRL Road Note 29 (mm)	AASHTO Pt 2.5 (mm)	Portland Cement Association (mm)
5	160	180	175
26	150	170	150
37	140	160	140

Pt Terminal Serviceability Index.

and horizontal alignment and Table 2 summarises the designs of the trial sections. Each section was designed to be 400m in length and 7m wide, with 1.5m surface dressed shoulders. Later changes and difficulties with sub-base levels caused some variations from Table 2, which are described below.

2.1 JOINT DESIGN

Expansion joints were not required because:

a) none of the test sections was butting against a fixed structure and

b) construction took place during a relatively hot season.

Contraction joints were specified at either 5 metre intervals, or a mean of 5 metres ranging from 4.5 to 5.5, in order to compare the drumming effect from vehicle tyres passing over regularly spaced and random spaced joints. For the same reason, some joints were cut perpendicular to the centre line of the road and some skewed by 10°. Aggregate interlock across the joints was considered to be sufficient to prevent stepping under the anticipated traffic loading but the stronger section, C3, was given dowelled joints in order to determine whether this more rigid joint prolongs the life of joint seals.

The specification called for the contraction joints to be sawn as soon as the concrete was set sufficiently not to tear, and to be sealed with an elastomeric joint sealant. Possible alternatives included a hot poured bitumen, bitumen and sand mixture and, if narrow joints could be cut, no sealant at all. This was to establish if a cheaper specification could be justified for relatively light traffic.

Construction joints tied in the conventional manner with deformed steel tie bars 16mm in diameter at 600mm centres were specified for the end of each day's work or when a delay occurred. The same tie bars were used at 1m intervals at the warping joint between the two carriageway lanes. No groove or sealant was specified for tied joints. Contraction and warping joints are of the types shown in Figure 2.

2.2 CONCRETE MIX DESIGN

Slab thicknesses in Table 1 were calculated using a concrete characteristic flexural strength of 4MPa, or corresponding minimum compressive strength of 30MPa for Road Note 29 (RRL, 1970) and average compressive strength of 35MPa for the AASHTO design (AASHTO, 1974). This strength corresponded with Class 30 con-

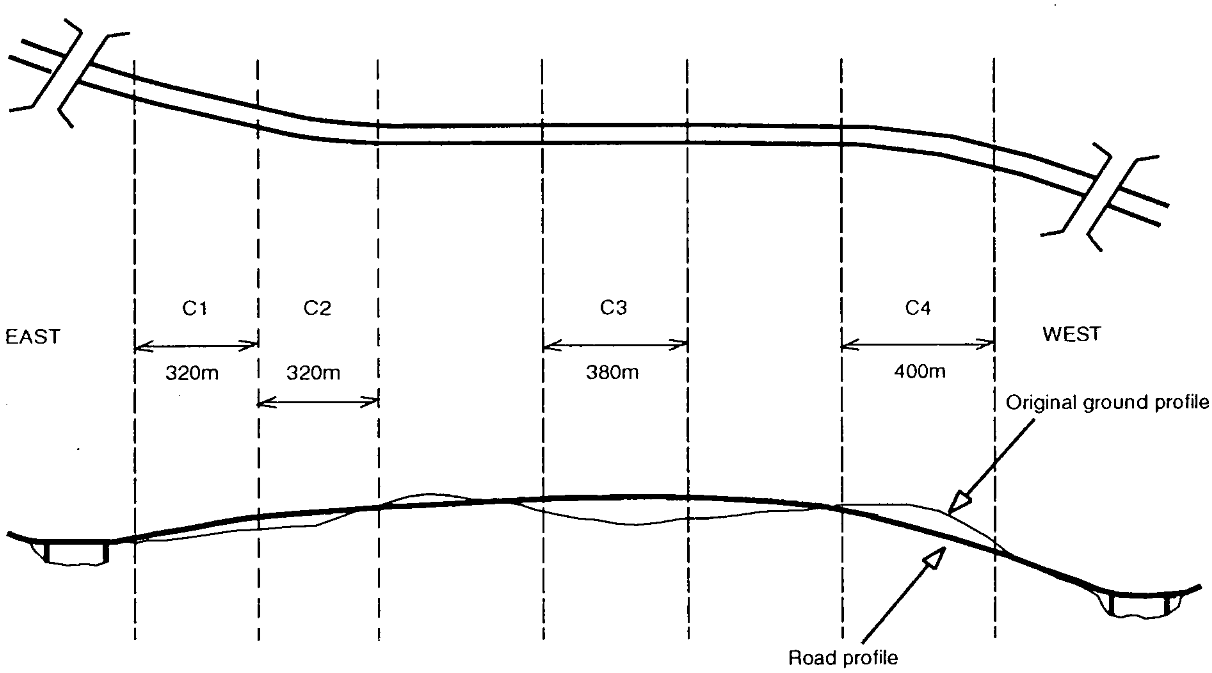


Fig. 1 Horizontal and vertical alignment of the trial sections

TABLE 2

Test section design

Section	Slab Thickness (mm)	Sub-Base	Subgrade	Remarks	Joints
C1	150	150mm Class 3,3 LS 93% Tar Prime	Imported 150mm SG 9	On 800mm Fill	10° SKEW @ 5m spacing
C2	125	150mm Class 3,3 LS 93% Tar Prime	Imported 150mm SG 9	On 800mm Fill	SQUARE @ 5m spacing
C3	175	NIL	Imported 150mm SG 9 Tar Prime	On 1m Fill	SQUARE Dowelled @ 5m spacing
C4	125	NIL	Imported 150mm SG 9 Tar Prime	On 3m cut. Levelled and compacted.	SQUARE variable spacing 4.5m-5.5m

Note: Class 3.3 - Natural gravel, Texas triaxial class 3.3 max
LS 93% - MOD AASHTO
SG 9 - Natural gravel of minimum CBR 9%

crete in the Zimbabwe specifications. Trial concrete mixes were prepared in the Zimbabwe MOT Test Laboratories using the proposed materials from sources near the site. These samples were tested for compressive and flexural strength at both the Zimbabwe laboratories and at TRL. Details are shown in the Appendix, Tables A3 and A4. The mix resulting from the trials is shown in Table 3 with the proportions actually laid.

3. CONSTRUCTION

The subgrades and sub-bases were prepared by the Ministry of Transport and the concrete was laid by a local contractor. There was no previous experience of constructing concrete pavements in the country, apart from a few hard standings, but this contractor had used concrete in building dams and reservoirs.

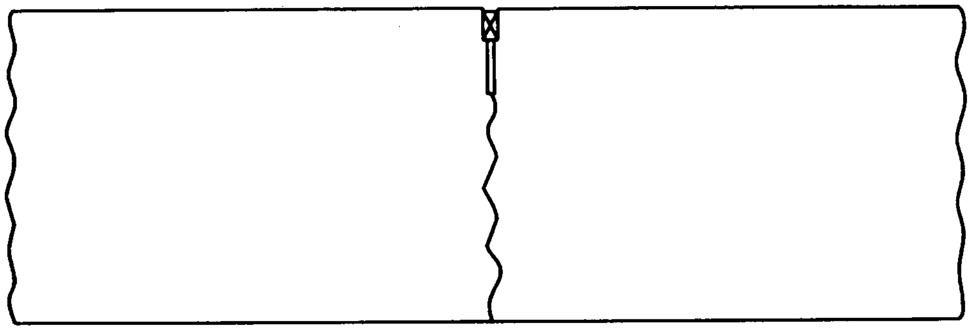
3.1 SUB-BASE AND SUBGRADE MATERIALS

Sections C1, C2 and C3 were built on low embankments but section C4 was in cut (Figure 1) and it was found while cutting the profile of section C4 that the subgrade level at one end of the section was on very weak clay over a length of 80m. This material was removed to a depth of 400mm and was replaced with gravel taken from the ditches at the other end of the same section. This was compacted and covered with approximately 150mm of imported sub-base material.

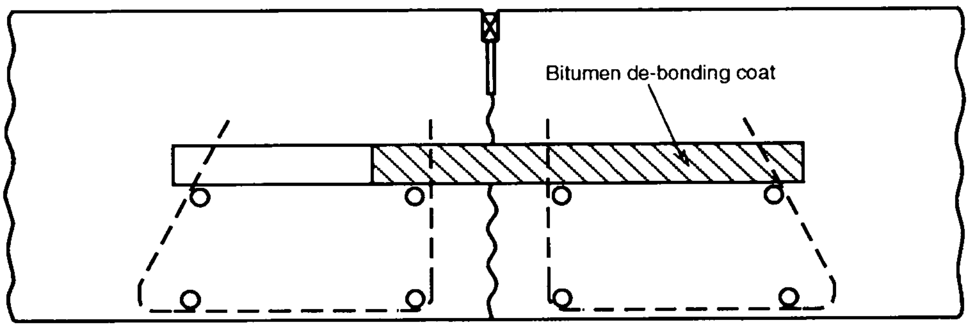
The sub-base material was generally well graded and conformed to the standard specified in almost every respect, Table A5. However, the fraction passing the 75 micron sieve was at the top of the permitted limit, or just above it. This and the entry of water between the shoulder and concrete slabs, or through damaged joint seals, has led to clay fractions being evacuated from beneath the concrete by downward or rocking movements of some of the slabs. This process is termed 'pumping'.

The subgrade material used under the sub-bases on sections C1 and C2 and directly under the concrete on sections C3 and C4 was also well graded, apart from the oversize material which was mostly rejected by hand. But this material also had a high fraction passing the 75 micron sieve and has been eroded in a similar manner on sections C3 and C4.

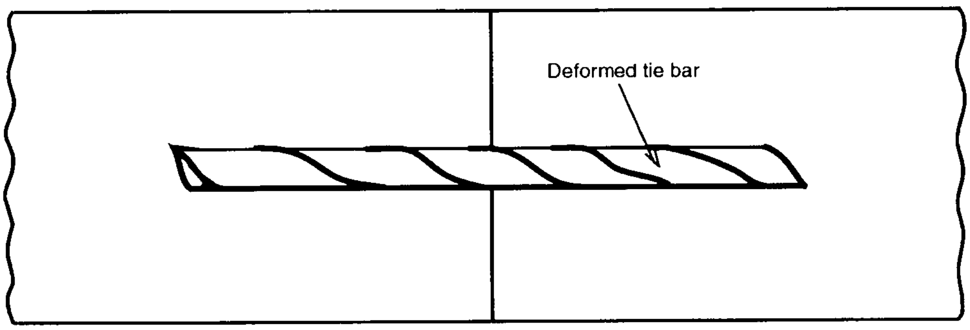
Day-time rocking of many concrete slabs shows that, although both of these materials exhibit high strength in terms of in-situ CBR (Table 4), they have proved to be inadequate for use as sub-bases under a jointed concrete pavement where water is permitted to enter the pavement structure. A limited amount of mud staining has been recorded during the visual surveys on the shoulders of all the sections (Figures A1 to A4) but is most apparent at the end of section C4 (Figure A4) at the lower side of a super-elevated section where rain water has drained across the road and penetrated the gap between pavement and shoulder as well as entering by damaged transverse joint seals. Plate 1 shows the most pronounced example of mud staining on the shoulder due to fine material being pumped up through damaged joints



Contraction joints on Sections C1, C2, & C4 without dowels



Contraction joints on Section C3 with dowels



Warping joints on all sections

Fig. 2 Contraction joints and warping joints

TABLE 3

Concrete mix proportions

	Aggregate proportions (%) stone	Aggregate proportions (%) river sand	Aggregate proportions (%) pit sand	Cement/ aggregate (%)	Water/ cement (%)
Recommended:	63	32	5	19	54
Laid:	63	32	5	22.4	47.5
	64	29	7	22.7	45

and through the gap between the shoulder and the concrete slabs. There is no apparent correlation between the amount of pumping, according to stains on the shoulders, and the magnitude of the measured deflections, but staining was more marked at the lower side of all three super-elevated sections.

The loss of fine material from the sub-base results in voids under the concrete but is difficult to measure because of the effects of slab warping, which are discussed below in Section 4.7. However, it is considered that this is one of the causes of the accelerated cracking of the slabs. Road Note 29 (RRL, 1970) states that no sub-base is required under concrete slabs for subgrades of CBR greater than 15 per cent. However, the current

TABLE 4

Sub-base and subgrade strength testing by DCP.

Section	CBR % Average	Range
C1 Sub-base	52	33 - 86
C2 Sub-base	72	36 - 125
C3 Subgrade	43	30 - 70
C4 Subgrade	62	41 - 108

Departmental Standard, 'Structural design of new road pavements' (DTp, 1987) specifies a cement-bound or lean concrete sub-base for all rigid pavements in order to prevent pumping from the sub-base and, in most cases, from the subgrade too.

3.2 CONCRETE ROAD-BASE

A water tower was set up on a site adjacent to the trial sections with stockpiles of the coarse and fine crushed stone, river sand and pit sand. These were fed through a weigh-batcher to a half-cubic-metre mixer which discharged into a lorry-mounted skip. When five or six loads had been discharged into the skip, it was driven to the laying site and tipped between rolled steel channel forms while a second skip was positioned to receive the concrete from the mixer. Samples of the fresh concrete were taken three times a day for wet sieving analysis and

Plate 1 Pumping stains on the hard shoulder of test section C4

to fill cube and beam moulds. The rest was spread manually between the forms. Several vibrating pokers and two vibrating screeds were used to compact and level the concrete, which was then textured using a purpose-made broom and covered with wet hessian laid over wooden frames. After a few hours the frames were removed for use on fresh concrete and the hessian was laid directly on the surface. The following day the hessian was replaced with sand, nominally 50mm deep, which was sprayed with water.

Most of the joints were cut with a diamond wheel within 24 hours of laying. All were cut within 48 hours but some shrinkage cracks between the joints became apparent within days of laying; these are marked on the visual condition reports, Figures A1 to A4.

When the contractor had left the site, the Ministry of Transport constructed the shoulders with cement-stabilised natural gravel and sealed them with a bituminous surface dressing.

3.3 CONCRETE CHARACTERISTICS

During the construction of the trial sections a record was kept of all mix proportions and the times between mixing, finishing and joint cutting. Notes were made of all incidents such as delays and breakdowns, and a record was kept of weather conditions, including air temperature and wind speed, as possible causes of variability in the quality of the finished concrete.

The cube and beam specimens made at the laying sites were tested at the Central Test Laboratory in Harare. The results are listed in the Appendix in Tables A6 and A7. After the concrete construction was complete, cores were cut from the pavement for verification of concrete thickness, mixture quality and strength. These crushing strengths are listed in Table A8. Wet sieving results are shown in Figure A5 with the recommended grading envelope for standard class 30 concrete in Zimbabwe.

The test results on cubes and beams made at the time of construction show that in most cases the concrete meets the specification described in Section 2. The 90-day equivalent cube strength of the cores cut from the pavement also indicate that the average strength was at least as high as that specified, even taking into account strength gain from 28 to 90 days, which was in the order of 10%.

The recommended cement/aggregate ratio resulting from mixture trials prior to construction was 19% (Table 3). The contractor used a ratio varying from 22.4% to 22.7%. This resulted in higher strength but may also have contributed to early shrinkage cracking by increasing the heat of hydration.

Plate 2 shows typical cores cut from section C3. Compaction is seen to be uniform and distribution of aggregates is also good. The contractor achieved the same concrete density in the pavement as in the samples made for laboratory testing.

Plate 2 Cores cut from test section C3

3.4 CONCRETE SLAB THICKNESS

The surface of the trial sections conforms closely to the design vertical alignment. However, the sub-base on sections C1 and C2, and the subgrade on sections C3 and C4 were not finished as accurately. The thickness of the concrete slab is the difference between the finished surface and the sub-base level (or subgrade in the case of C3 and C4) and varies according to the deviation of the actual sub-base level from the design profile. On all four sections, levels were taken at 10 metre intervals at five stations across the pavement, on the sub-base (or subgrade) and on the finished concrete. These are shown in OU WP210. Table 5 summarises the readings and shows that, instead of the three thicknesses of trial pavement as intended in the original design, there is a variation of slab thickness ranging from 80mm to 237mm over the total 1.42km. The variation in thickness of individual slabs is typically 15-20mm. This larger variation, although not intended, will provide more scope to identify the effects of thickness on the various modes of deterioration.

4. POST CONSTRUCTION CONDITION MONITORING

4.1 VISUAL CONDITION SURVEYS

Each year since 1987 a visual condition survey has been made of the trial sections, in a similar manner to that described in the UK DTp Standard HD17/88 (1988). On schematic maps of each section, defects such as cracking, joint damage and pumping stains on the shoulders, have been marked and later summarised in tables. Figures A1 to A4 contain the condition assessments made in 1992 and Tables A9, A10 and A11 show the progression of deterioration from 1987 to 1992 of the joints (see 4.2 below) and the slabs (see 4.5 below). The condition assessments in Tables A9, A10 and A11 are subjective and consequently show some anomalies, particularly with joint sealant condition and transverse cracking.

TABLE 5

Test sections - as built.

Section:	C1	C2	C3	C4
Slab Thickness 'As built' Average:	173 mm	124 mm	186 mm	151 mm
Range:	135-213 mm	80-189 mm	139-237 mm	96-233 mm
Design thickness:	150 mm	125 mm	175 mm	125 mm
Joint spacing:	4.5 - 5.5 m	5 m	5 m	4.5 - 5.5 m
Joint type:	10° Skew	Square & 10° Skew	Square	10° Skew
Load transfer:	-	-	Dowels	-
Sub-base:	150mm Class 3.3 LS 93%	150mm Class 3.3 LS 93%	Nil	Nil
Subgrade:	Imported 150mm of SG 9	Imported 150mm of SG 9	Imported 150mm of SG 9	Imported SG 9 + Tar prime
On Fill or Cut:	On 800 mm Fill	On 800 mm Fill	On 1 m Fill	On 3 m cut
Chainage:	17.480 -17.800 km	17.800 -18.120 km	18.580 -18.960 km	19.360 -19.760 km
Length:	320 m	320 m	380 m	400 m
No. of slabs:	128	128	152	163

Note: Class 3.3 - Natural gravel, Texas triaxial class 3.3 max
LS 93% - MOD AASHTO
SG 9 - Natural gravel of minimum CBR 9%

4.2 JOINT CONDITION

The criteria used to judge sealant condition was:

GOOD - No visible damage

FAIR - Some loss of adhesion but still effective

BAD - The joint allows water to pass through to the sub-base or subgrade.

Transverse cracking close to joints can be classified as joint spalling when further deterioration takes place.

Table A10 shows most joints to be properly sealed in 1988, the worst section, C4, having 2% of joints in bad condition and 24% showing signs of aging but not yet letting in water. However, 350 metres of joint sealant was replaced in 1987. Apparently the locally manufactured material was applied properly in some areas but not in others. This figure includes 70 metres of bitumen/sand filler, which was not successful and was replaced.

Joints in concrete pavements that are built on foundations with little susceptibility to water damage need not be repaired until they deteriorate beyond the "fair" state to "bad". The sub-base material under sections C1 and C2 and the subgrade under sections C3 and C4 have both been eroded by water penetrating the joints so the bad and fair joints should be renewed. It would also be economical to reseal the joints currently in good condition at the same time, because their remaining life is very likely to be short after seven years of service.

A design feature that increased the penetration of water past the damaged joint seals is the skew of the joints on test sections C1 and C4. Plate 3 shows how water running down the texture grooves is intercepted by the skewed joint and penetrates the joint far more easily than it could if the joint were cut parallel to the texture grooves. Clearly, the more water that penetrates the joint, the more likely that damage will occur to the sub-base or subgrade material.

It is arguable that very little water could penetrate poorly sealed joints that are installed on pavements parallel to pronounced drainage grooves, as shown in the upper photograph of Plate 3, irrespective of whether the grooves and joints are set perpendicular or skew to the centre line. On gradients, the grooves and joints would be better set skew in order to follow the line of greatest slope.

4.3 ROAD ROUGHNESS

Road roughness was measured once a year in each wheelpath using the TRL Profile Beam (Abaynayaka, 1984). The results, averaging near-side and offside measurements, are given in Table A12 and illustrated in Figure 3.

Although road roughness, or riding quality, is only one indicator of pavement condition, an increase of roughness with time is an important indicator of pavement deterioration (Smith et al 1980). On concrete pavements this increase is caused by uneven settlement of slabs or broken parts of slabs, faulting at joints, or severe spalling at joints or cracks. The roughness measurements to-date indicate no serious deterioration, although a few cases of faulting are recorded on trial section C4 (Figure A4).

Slab warping, which is discussed in more detail in Section 4.7, has a small effect on the longitudinal profile. However, a jointed pavement with slabs 5m long i.e. 200 slabs per kilometre, each warping by approximately 1.0mm during the daytime hours of measurement, would register a maximum roughness change of about 200mm per kilometre, which is little more than the variability of measurement.

4.4 SKIDDING RESISTANCE AND TEXTURE DEPTH

On surface dressed and asphaltic concrete surfaces, texture depth frequently affects the resistance to skidding. This is not the case on Portland cement concrete surfaces with texture of the type used on the trial sections and illustrated in several of the plates. Skidding resistance is primarily dependent on the micro-texture of the surface (Franklin, 1988), which in this case is formed by the sand particles in the concrete. The macro texture is dominated by the drainage grooves but on this type of surface, macro texture only affects skidding resistance under conditions of heavy rain.

Skidding resistance was measured using the TRL skidding resistance pendulum according to Road Note 27 (Road Research Laboratory, 1969). Measurements were made in both wheelpaths at three fixed locations in each direction on each trial section. Table A13 summarises the results and Figure 4 illustrates the measured variations.

The results suggest that there has been a decrease in the skidding resistance value over the years 1990 to 1992. However, there is a considerable fluctuation in the readings recorded from year to year, primarily due to changes in operator of the skidding resistance tester rather than real changes due to traffic and climatic influence on the road surface. The uncharacteristically high readings for the year 1990 appear to exaggerate this decrease. Road Note 27 (1969) recommends a minimum skidding resistance value of 45 for straight roads with easy gradients and curves, and a value of at least 65 for gradients above 5%, which would include part of trial section C4. All the measurements recorded are in excess of these figures.

No reduction of texture depth, as measured by the sand patch method, can be identified from Figure 5. This is because, although some of the micro texture may have been worn away, there has not been enough traffic on the sections to expose the coarse aggregate or to wear down the material between the drainage grooves. These grooves, shown in Plate 4, were set at an average pitch of 25mm and vary in depth from 0 to 7mm, averaging

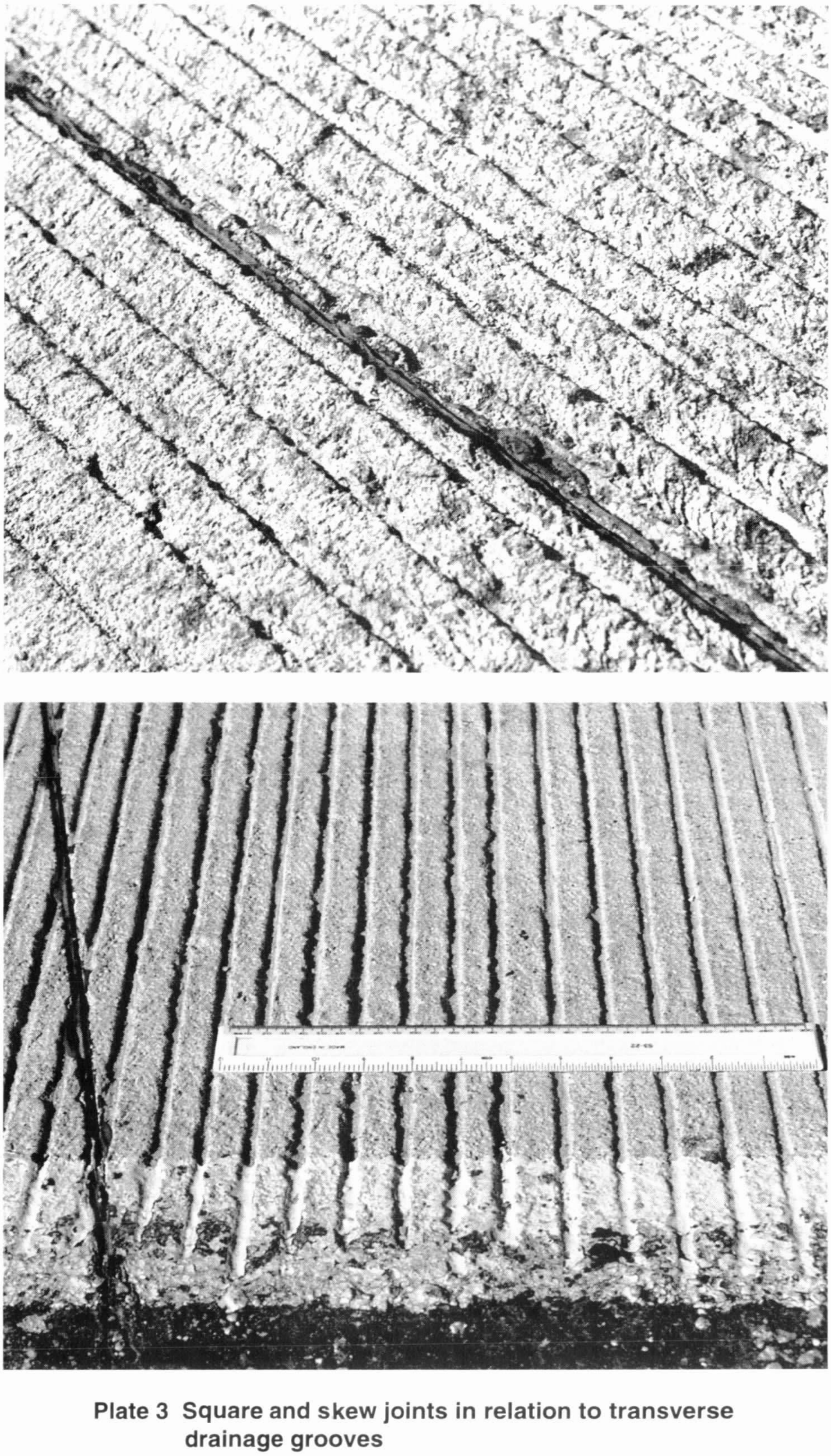


Plate 3 Square and skew joints in relation to transverse drainage grooves

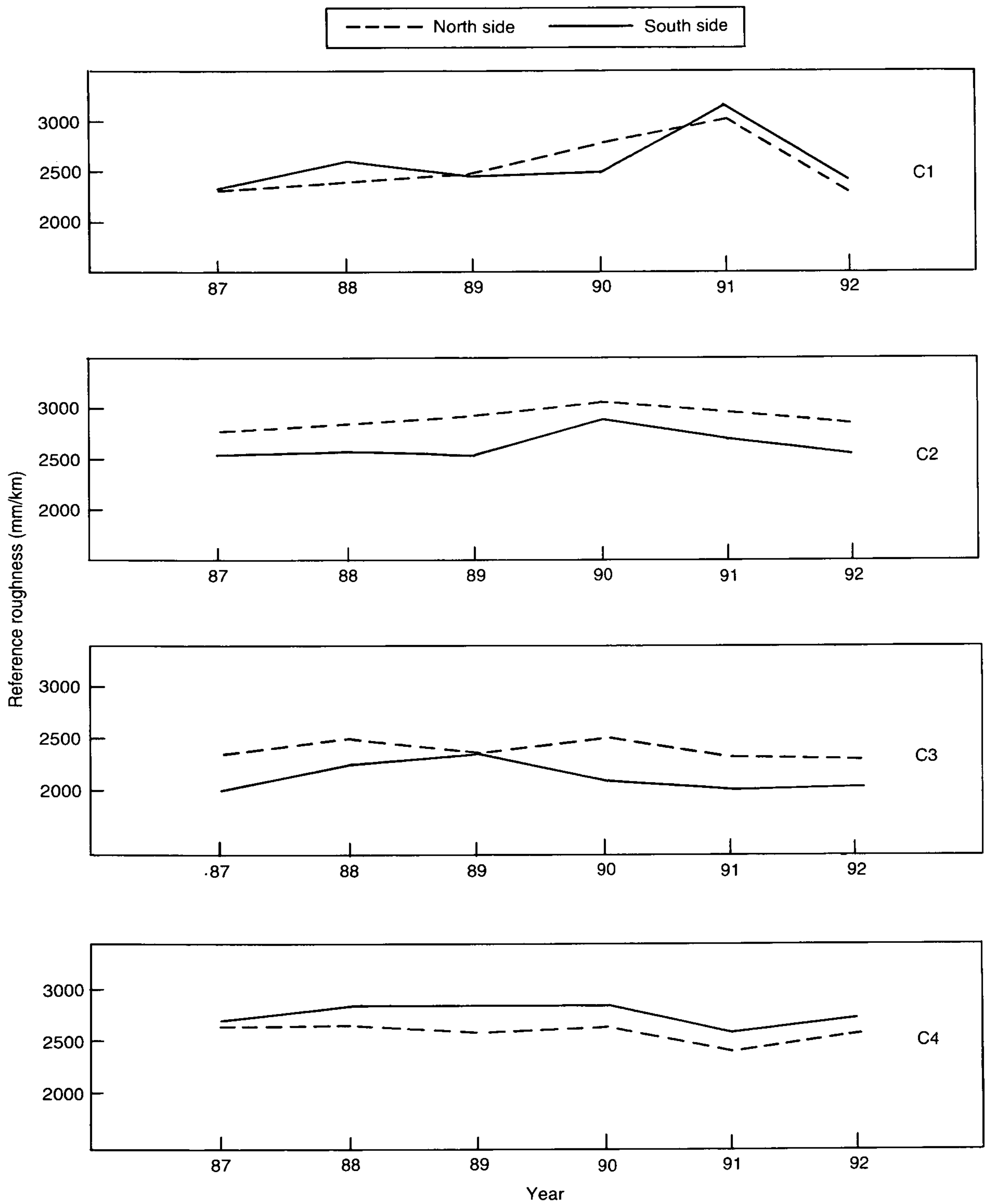


Fig. 3 TRL profile beam roughness measurements

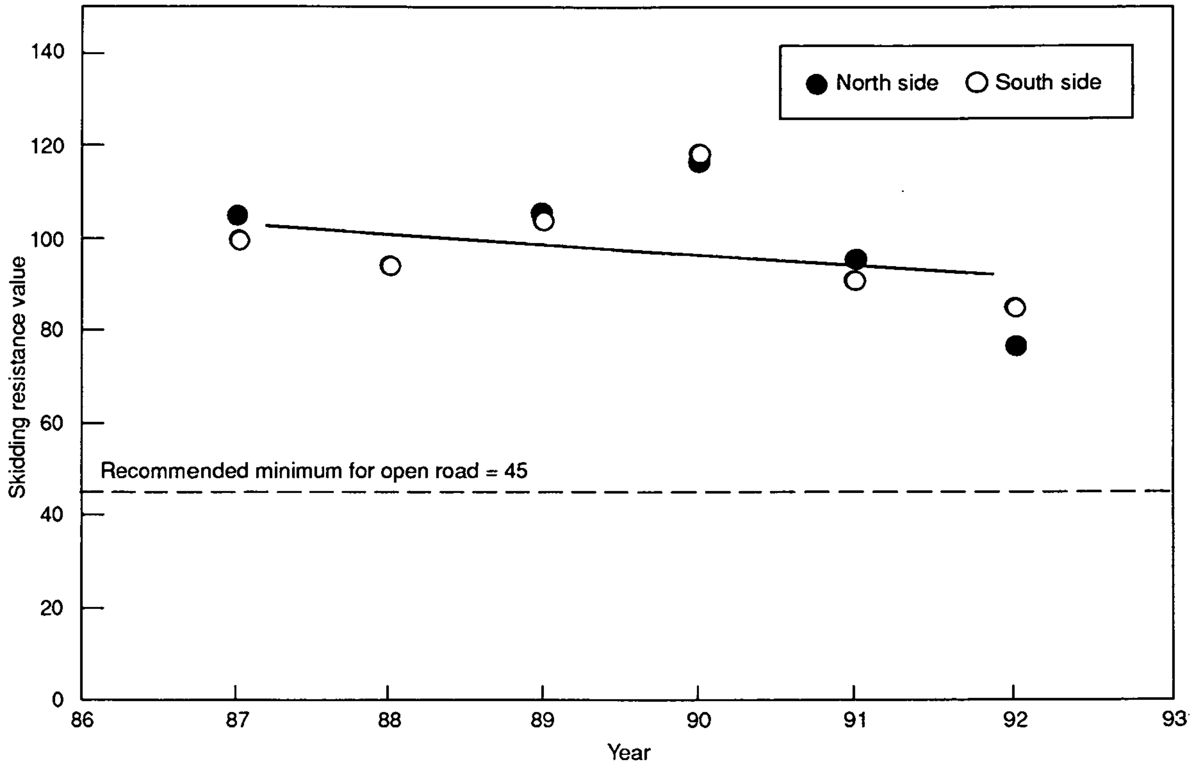


Fig. 4 Average skidding resistance (all sections)

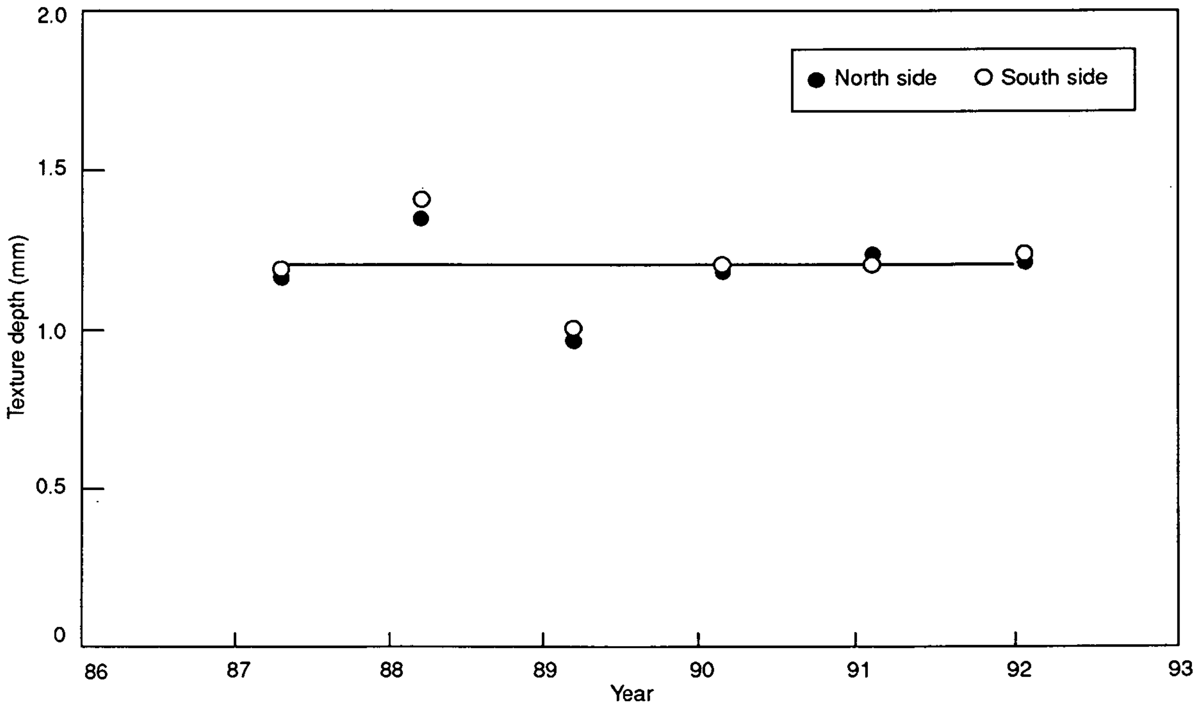


Fig. 5 Average texture depth (all sections)

Plate 4 Surface texture

about 3mm. Table A14 summarises the texture depth measurements made during each dry season.

Some concrete surfaces have been criticised for tyre noise, mostly on surfaces with pronounced texture grooving, as on these test sections. No noise measurements have been made but subjective tests suggest that the tyre noise on the concrete text sections is less than that on the adjacent flexible test sections, which are surface dressed with 19mm stone. It is thought that randomising the pitch of the grooves, as shown in Plate 4, contributes to the reduction in noise level compared to that on pavements with regularly spaced grooves. No change in tyre noise has been detected subjectively when driving from square to skew jointed sections or from slabs 5m long to slabs of random lengths between 4.5 and 5.5m. This is probably because these differences are masked by the effects of the texture grooves and because stepping is not prevalent.

4.5 TRAFFIC LOADING AND SLAB DEFECTS

Traffic using the trial sections has increased since the completion of the upgraded road link in 1989. The present trend with modest further increases would bring the 20 year traffic loading close to 1 million ESAs, as estimated at the design stage.

The concrete was of good quality (section 3.3) and defects were few. Of the 61 metres of transverse cracking recorded in Table A9 during the first survey in 1987, most were due to shrinkage immediately after construction, while the concrete was still weak. This level of defect is not unusual for 2.84 lane kilometres of concrete construction, considering the severe drop in ambient temperature during the night at that season of the year.

Tables A1 and A2 illustrate the spectrum of traffic using the test sections. The range of axle loads is typical for the country with a small but significant number exceeding 10 tonnes (the legal axle load limit is 8.2 tonnes). Table A1 includes an estimated additional loading during 1987 and 1988 due to vehicles carrying stone from a quarry to a road construction site. These lorries were granted a dispensation to exceed the legal limit and they were known to have axle loads in the region of 14 tonnes but were not travelling during the period of any surveys. They passed over the north lanes of the test sections in a fully loaded condition and are thought to have caused two transverse cracks on section C2 and one on section C4. At these locations the slabs were 110mm and 115mm thick (slabs C2 16 and 18 and slab C4 22 in Figures A2 and A4).

The UK Department of Transport Advice Note HA35/87 (1987) states "The failure condition (of a jointed concrete pavement) represents the end of the initial serviceable life, and the point where the rate of cracking begins to increase rapidly. At that stage the life of the pavement might be extended by suitable strengthening measures rather than excavation and complete reconstruction."

The failure condition of individual slabs is described as a crack across the width or length at least 0.5mm wide accompanied by a partial loss of interlock, or more than 200mm of cracking at least 1.3mm wide. Up to 30% of slabs can be expected to have reached the failure condition during the design life of the pavement.

The last column in Table A9 shows the number of "failed" slabs according to the above definition. Most of this damage occurred before the road was opened to traffic and is in the form of transverse cracking. However, the Table also indicates that further damage has been registered in the 1992 survey and repairs are required to several slabs as detailed in Chapter 5 below.

It should be emphasised that the "failed" slabs noted in Table A9 in no way affect the vehicles using this road. However, there are defects connected with the joints, i.e. spalling and stepping, that also require attention, (Table A11). Plate 5 shows a joint with spalling problems from the time of construction. The type of repair required is referred to in the DTp advice note HA35/87 (1987) as "arris or thin bonded repair." Where transverse cracks completely cross a slab close to the joint, full depth repairs are appropriate. Where a narrow transverse crack crosses in the central region of a slab without stepping, no repair is required.

Plate 6 illustrates shallow plastic cracking caused by drying winds blowing over the wet concrete before the hessian was placed to cover it. This cracking is not structural. Plate 7 shows the small corner cracking that occurred mainly on section C3 and Plate 8 shows a typical transverse crack; this one on section C1 was caused by shrinkage before traffic loading started.

Plate 5 Spalled joint

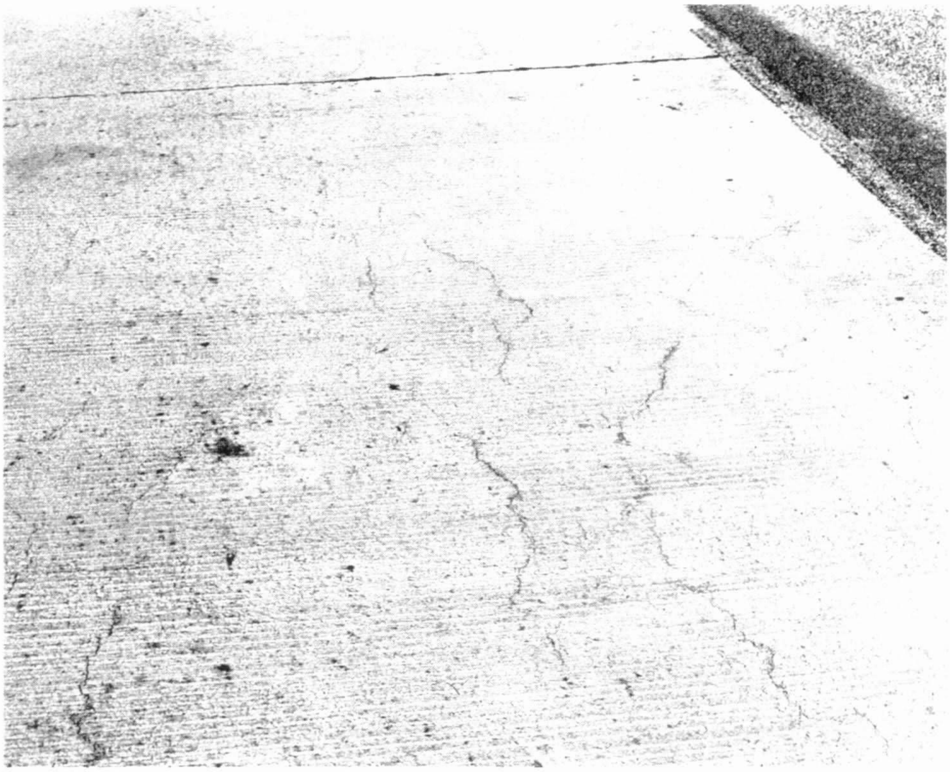


Plate 6 Plastic cracking on test section C3

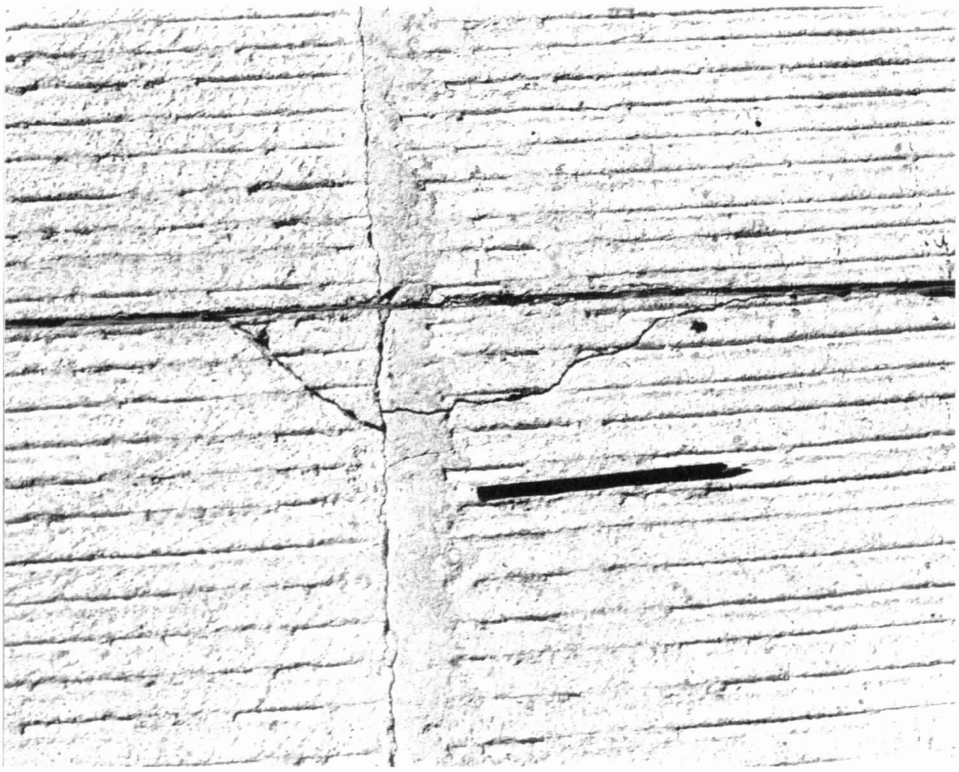


Plate 7 Corner cracking on test section C3

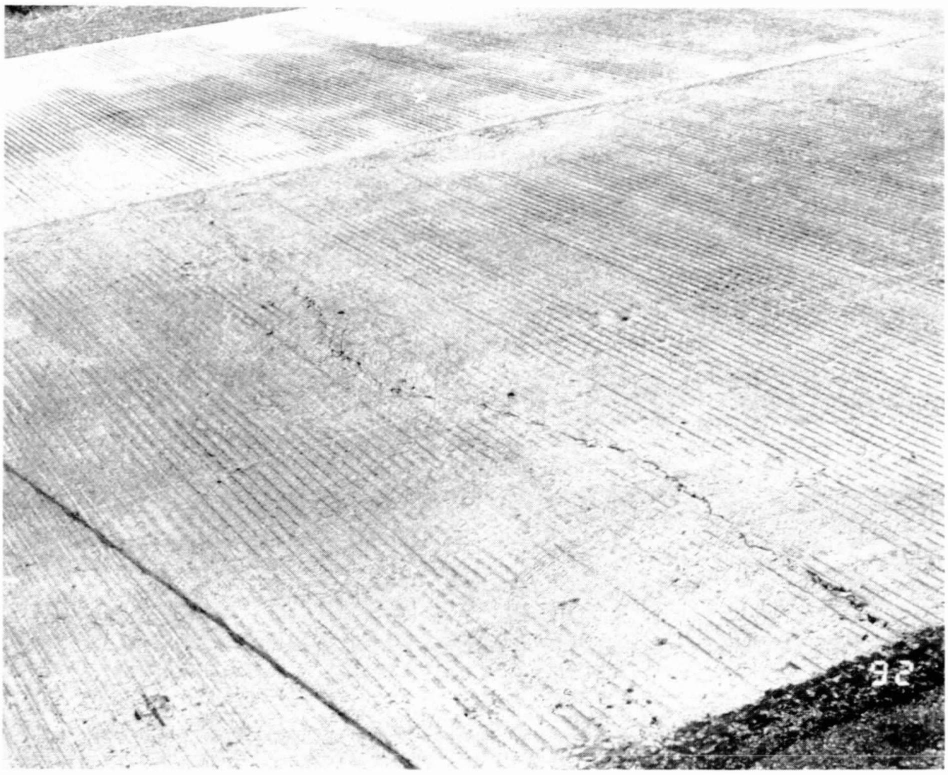


Plate 8 Transverse crack on test section C1

4.6 DEFLECTIONS

Starting in 1988, measurements of deflection were made using Benkelman beams and a lorry with a rear axle load of 6.36 tonnes. The aims were to measure interlock across the joints, compare deflections from year to year as a measure of pavement condition, and to detect slab rocking.

Figure 6 shows the initial position of the beams and lorry rear axle before it was driven forward. Records were made of the initial gauge reading, the maximum reading, and the final reading after the lorry had passed at least two slabs beyond the joint being examined.

The Benkelman beam readings are influenced by a complex combination of concrete flexural strength, sub-base compressive strength, interlock across the joint and, in the case of the off-side wheel, the influence of the central warping joint. Furthermore, when used in the traditional manner, the feet of the beams in the starting position rest on the concrete slab well within the influence of the lorry rear axle. This makes the calculation of actual deflections quite difficult since corrections are necessary.

Benkelman beams are too long to be used conveniently with their measuring probes resting on the corners of the slabs and with feet resting on the shoulder outside the influence of the lorry axle. In order to eliminate the influence of the lorry axle on the feet of the measuring instrument, a deflection logger was developed and built in the TRL workshops (Plate 9). This instrument uses a shorter arm and two linear transducers to measure vertical movement each side of a joint at the slab edge. Instead of the three readings recorded manually, (initial, maximum and final) the logger records fifty readings from each transducer over a set time. These may be printed later and/or plotted to show curves of deflections on each side of the joint under examination. Figure A6 is a typical print and plot of readings each side of a joint.

A summary of the vertical deflections measured by the Benkelman beams and the logger over successive surveys is shown in Table A15 and plotted in Figure 7 for one section. Figure 8 shows a summary of all the logger readings taken in 1992 on trial section C1 south side. The plot shows maximum minus final values of vertical displacement, V, on both sides of each joint. Where the points V_1 and V_2 are close e.g. at the right side of the graph, interlock is still good. The loss of interlock at the other end may be due to a number of factors, including low temperature at the time of measurement and sub-base erosion. There is also a plot of $V_1 - V_2$ on each logger plot (Figure A6).

There appears to be a correlation in Figure 7 between measurements made by the verge side Benkelman beam and the logger but more data are required before this can be defined with confidence, because the deflection is a combination of bending and rocking which affect the two instruments in different ways. Furthermore, there are both similarities and disparities between repeated measure-

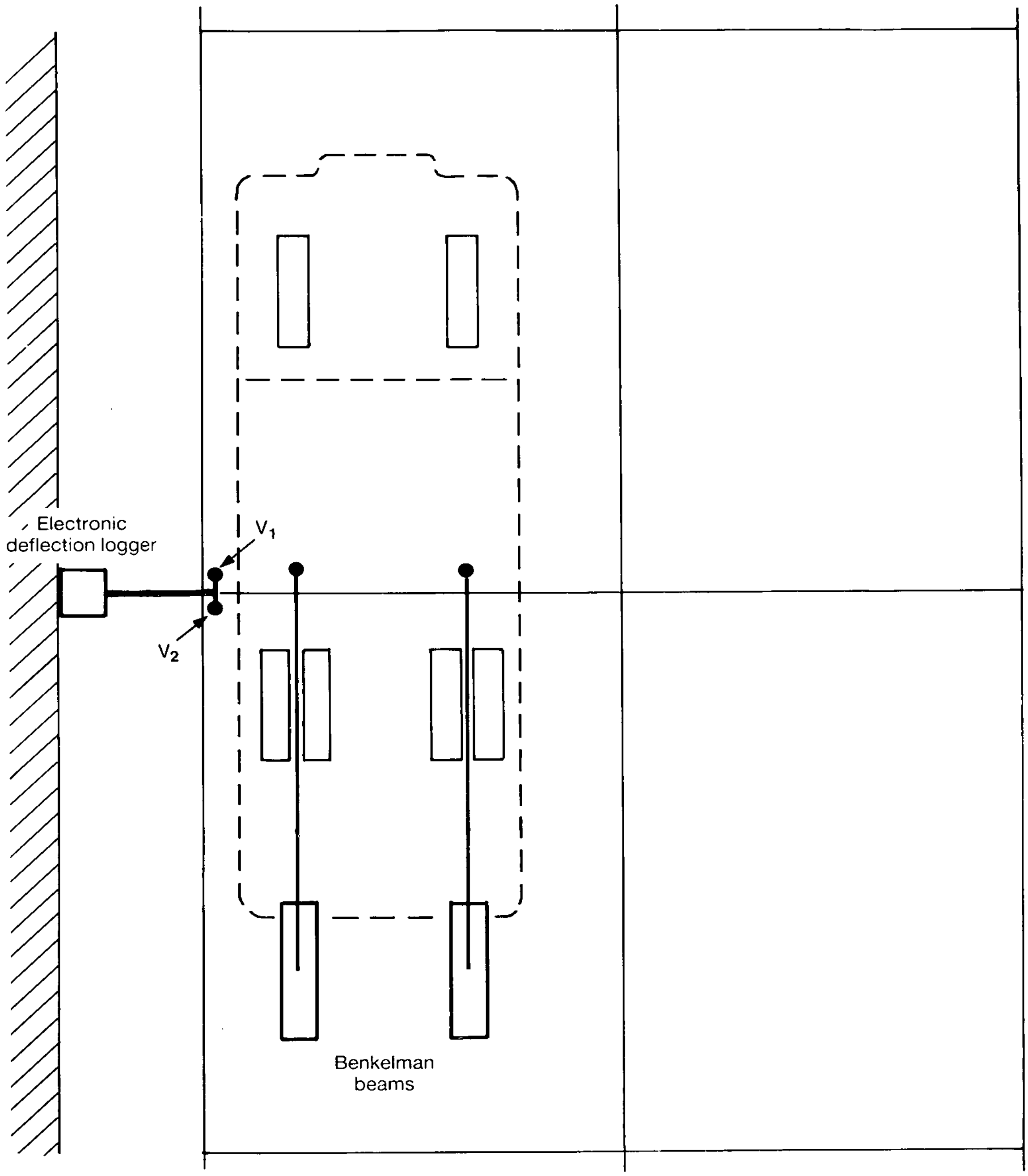


Fig. 6 Instrument positions for deflection measurements

ments made in 1992 on the same sections using the Benkelman beams (Figures 9 and 10). Examination of Figures 8, 9 and 10 led to the conclusion that weather conditions affect the measurements of deflections more than had been anticipated. This is discussed below in Section 4.7.

The work to date has revealed no change of deflection or of aggregate interlock between the annual surveys because the effects of linear expansion and warping, as discussed below, have masked any changes that have occurred. Attempts to match conditions of measurement from year to year, by repeating the measurements during the same month and hour of the day, have been frustrated by the strong influences of cloud cover and wind. The repeat measurements made in June and July 1992 shown in Table A15 and Figures 9 and 10 illustrate a) that average deflection measurements can be accurately reproduced under similar conditions and b) the disruptive effect on deflections of variations in radiation from the sun. Although it is expected that deflection measurements will increase with time due to erosion of the material beneath the slab edges, the averages of all readings taken each year show little evidence of this to date.

Plate 9a The TRL Deflection Logger

Plate 9b Close-up showing two vertical transducers and the gauge to measure horizontal strain

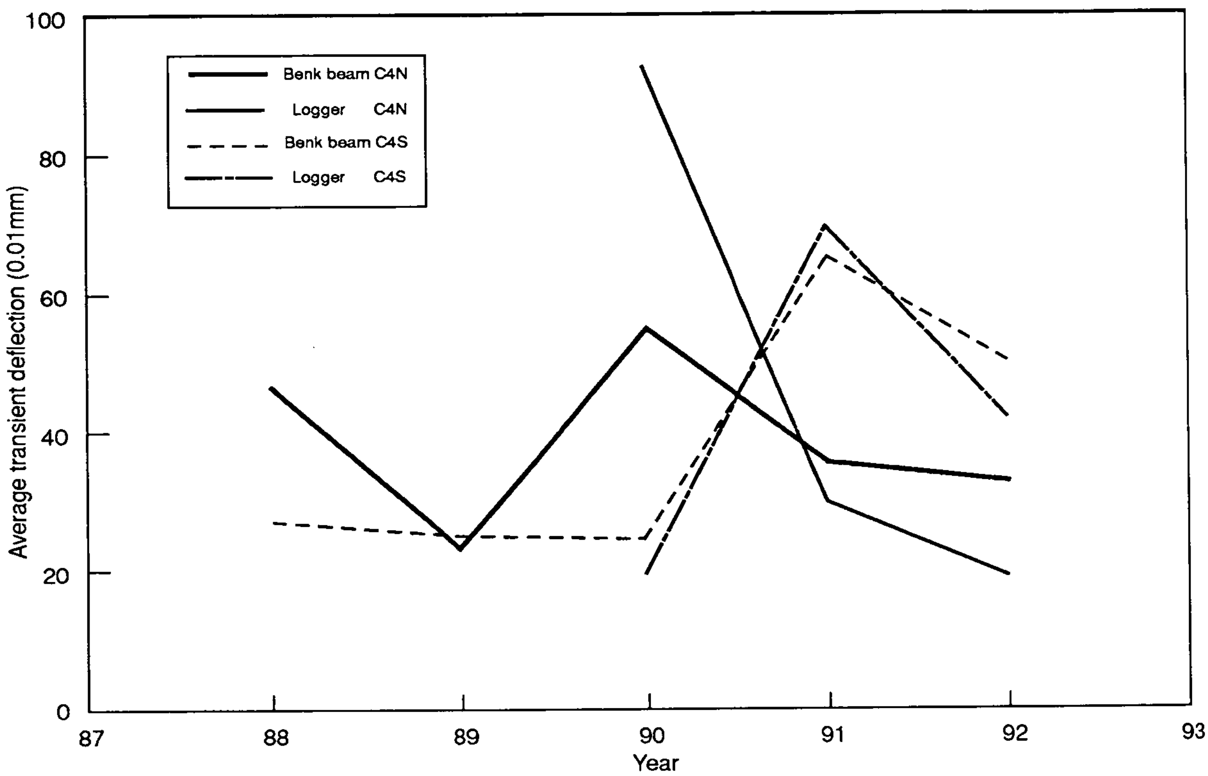


Fig. 7 Average transient deflections - Section C4

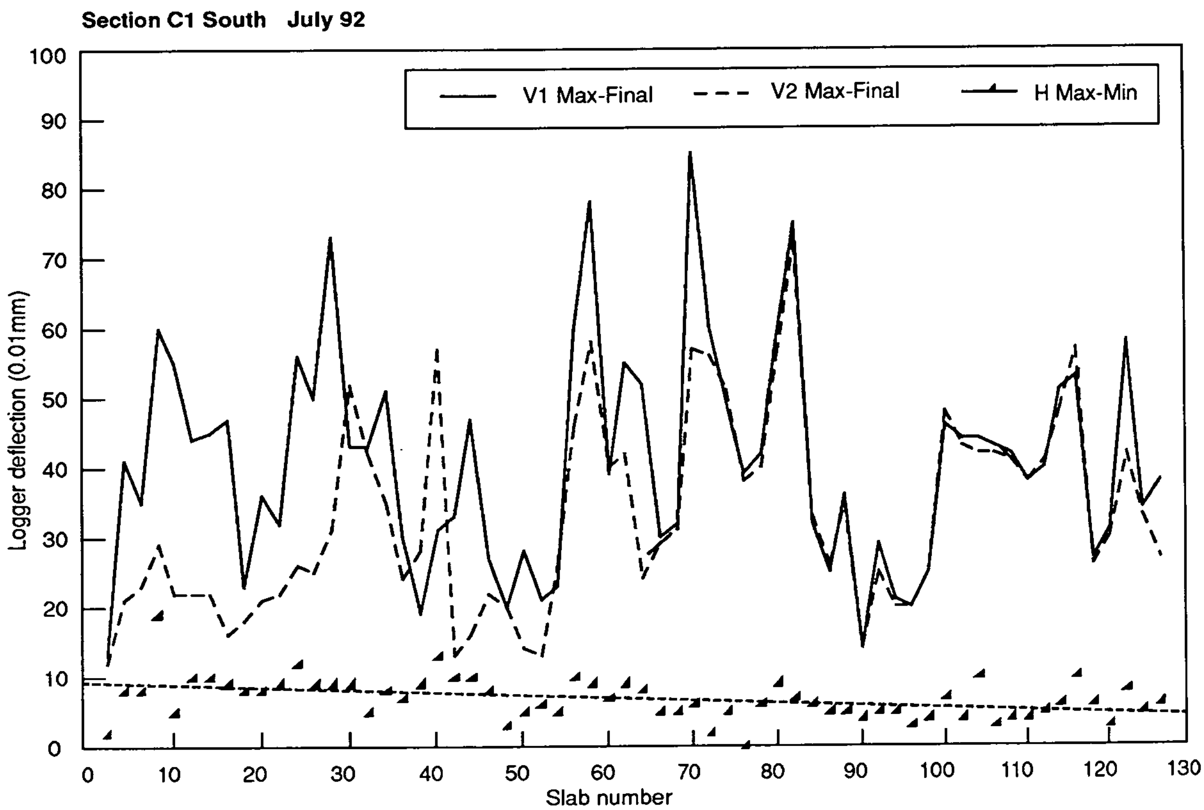


Fig. 8 Deflection logger readings

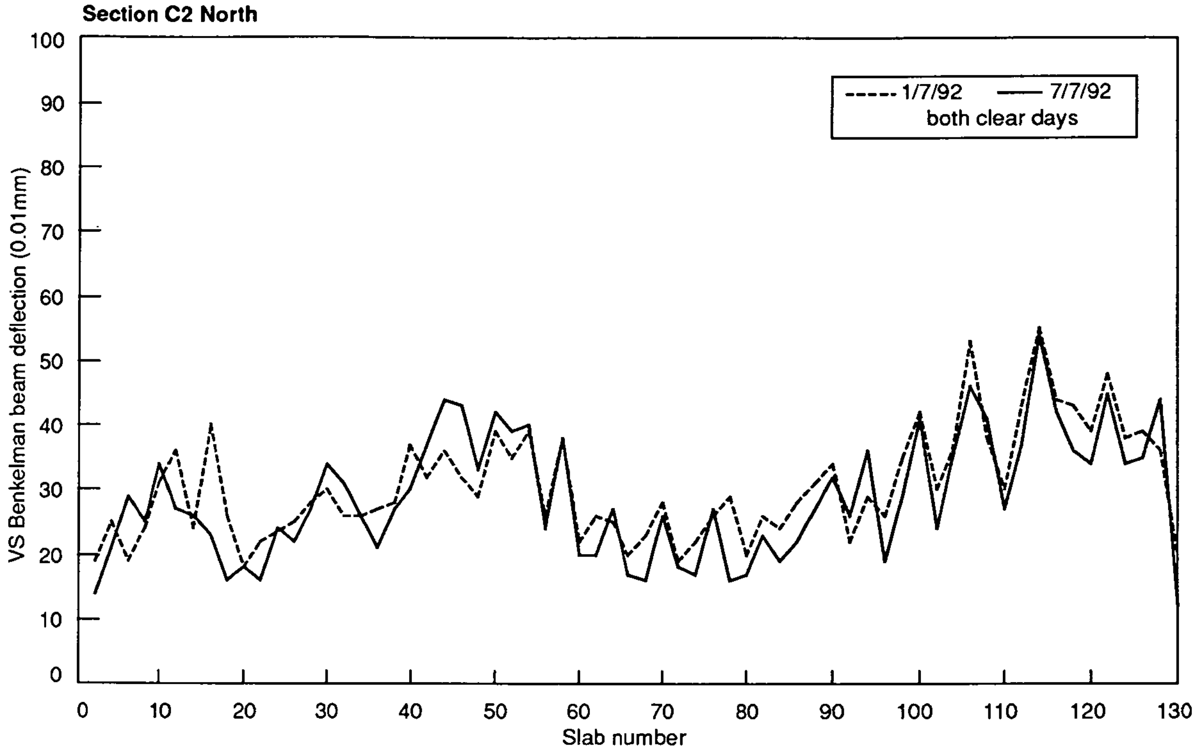


Fig. 9 Repeat beam deflection measurements - similar weather conditions

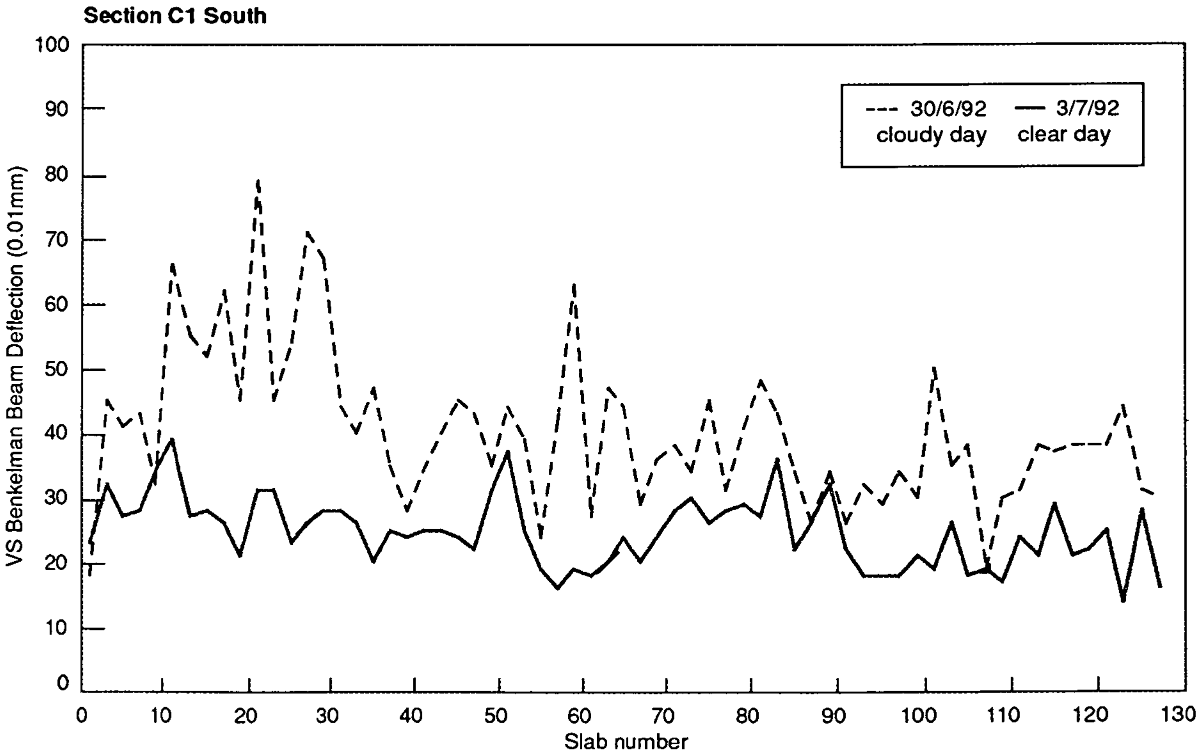


Fig. 10 Repeat beam deflection measurements - different weather conditions

Recently a third channel has been added to the logger circuits to record horizontal strain across a joint or crack in the pavement, measured by a modified Dmec gauge with a linear transducer set horizontally. This is shown hand-held in Plate 9. No conclusions have been drawn yet from these readings although there may be some significance in the effect shown in Figure 8 where the horizontal movement, H, is slightly larger on the left side of the Figure where the interlock is poor i.e. dissimilar vertical deflections V_1 and V_2 either side of the joints.

4.7 WARPING

4.7.1 Slab temperatures

The temperature at the top of a concrete road slab varies according to the weather and the time of day. The air temperature and rain have some influence but the largest effect is due to radiation, i.e. heat gain from the sun and heat loss to a clear sky at night. The temperatures at lower levels in the concrete vary less than at the top and follow the trend set by the top surface. In order to measure the effect of these temperature variations on the shape of the concrete slabs, and identify a connection between warping and pavement deterioration, levels were taken at nine points on the top surfaces of each of five slabs. The measurements were made on several days between sunrise and sunset with an optical level and a rod set into dimples on discs glued to the slabs in the positions shown in Figure 11. The height measurements and recording times for a typical slab are shown in Table A16. The temperatures at different depths in the concrete were also recorded over several 24 hour periods on two slabs of different thickness using thermistors and an automatic recorder. Similar temperatures were recorded on both slabs. The results are summarised in Figure 12 and Table A17. Detailed measurements of warping and temperature are listed in Overseas Unit Working Paper 289.

Figure 12 shows that over approximately eight hours during the day, the temperature is higher at the top of the slab than at the base; at night the reverse is true. This variation in the temperature differential causes differential linear expansion between the top and the base surfaces. The effect of this is a tendency towards hogging in the day time and dishing at night, as illustrated in Figure 13.

Figure 12 also shows that the temperature at the top surface is equal to that at the base at about 0900 and 1730 hours on a typically clear day. Whatever the actual shape of the slab surface at these times, for the purpose of analysis the slab could be considered flat; the day-time hogging and night-time dishing would then be relative to the actual shape at 0900 and 1730.

4.7.2 Warping measurements

Both transverse and longitudinal warping measurements can be derived from analysis of the levels taken at the nine points on a slab. Figure 11 shows three transverse paths and three longitudinal paths across the surface of a slab. The height of each centre point relative to a straight line drawn between the two end points for each path are plotted for one slab, the transverse paths in Figure 14 and the longitudinal paths in Figure 15, against time of

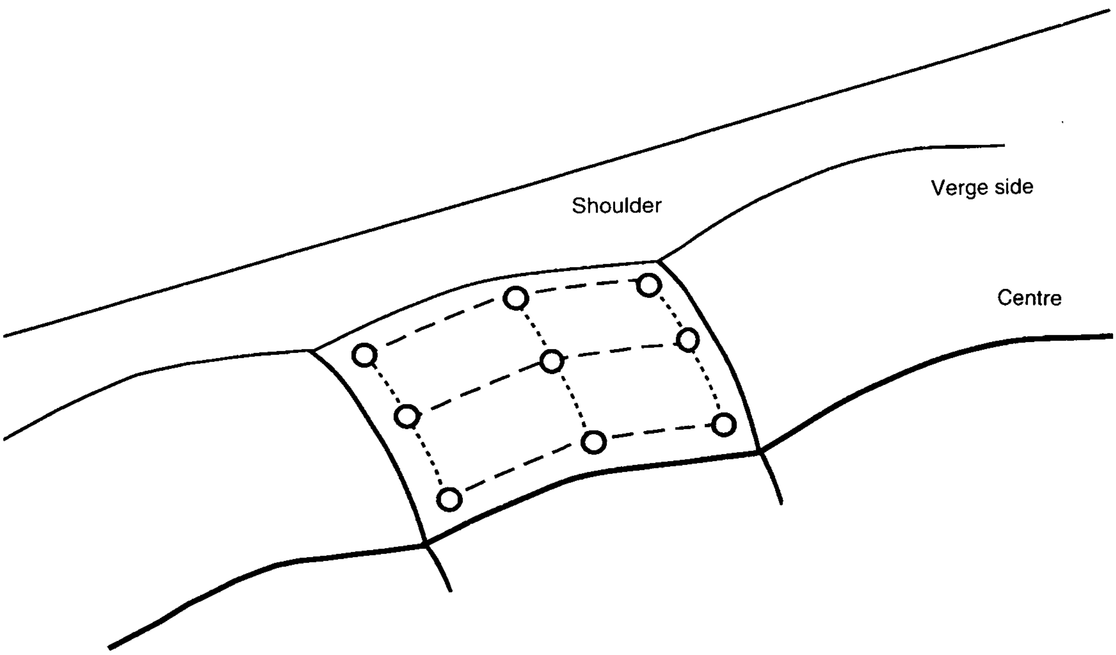


Fig. 11 Measuring points for slab warping

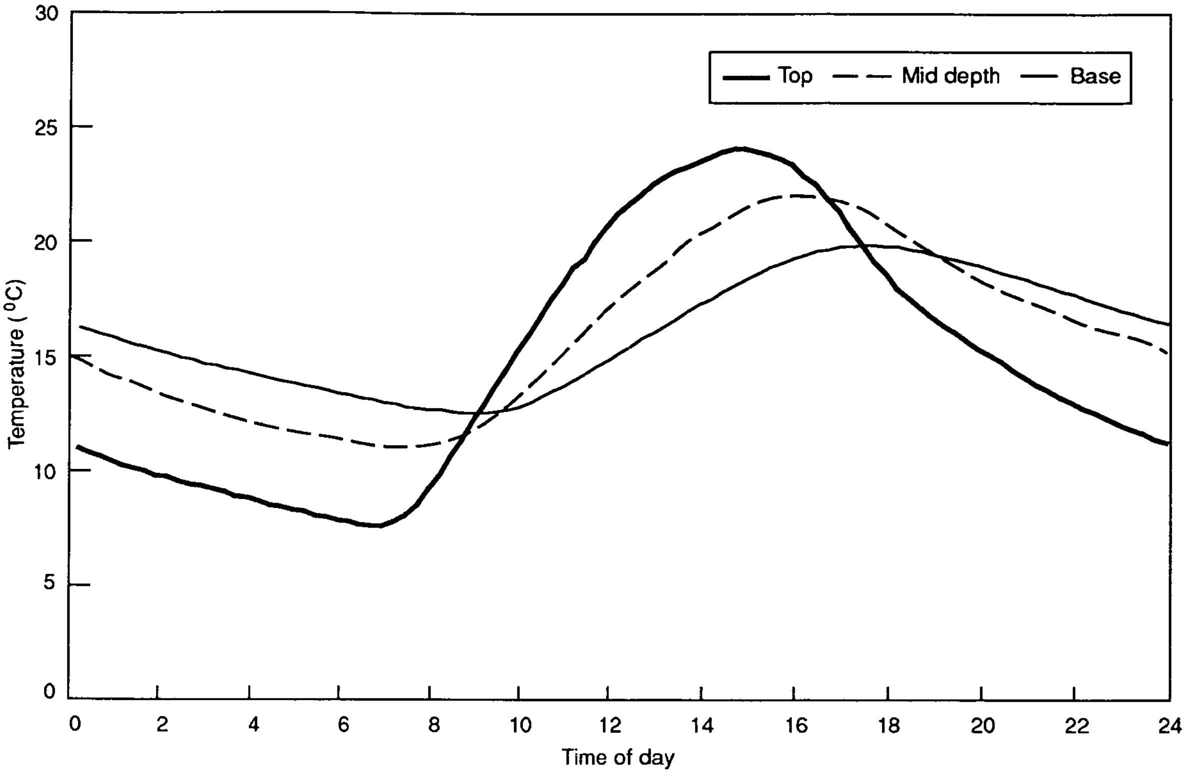


Fig. 12 Clear sky temperatures of slab C4-163

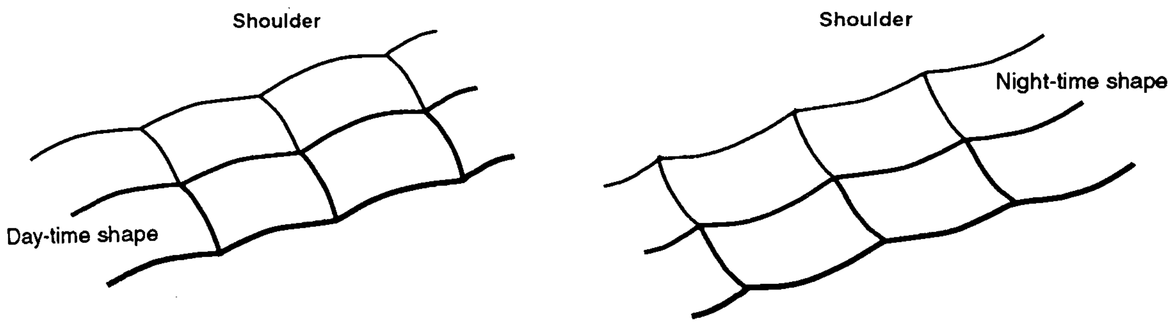


Fig. 13 Slab warping shapes

day. However, each concrete slab was cast with a slightly different shape so, in order to compare the different paths, all the heights were measured with respect to the value at 0900, when the temperatures on the top and bottom surfaces were similar. Figure 14 shows that the three transverse paths warp in a similar manner and Figure 15 shows for each of the three longitudinal paths a larger movement, but again all three paths are similar.

As the three transverse and the three longitudinal paths are similar, each group was aggregated to compare the warping on slabs of different length and different thickness. Figures 16 and 17 show very little difference in the amount of transverse warp or longitudinal warp on a slab 5m long and a slab 7.5m long. Figures 18 and 19 show similar warping of two slabs, one approximately 100mm thick and the other approximately 200mm thick.

To this point the analysis has considered only day-time movements. In order to calculate night-time effects, the temperature difference between the top and base of the slabs was plotted against longitudinal warping on two slabs 200mm thick, (Figure 20), and the resultant relationship was used to predict longitudinal warping over a 24 hour period. This predicted warping is illustrated in Figure 21 with measured results of day time warping of

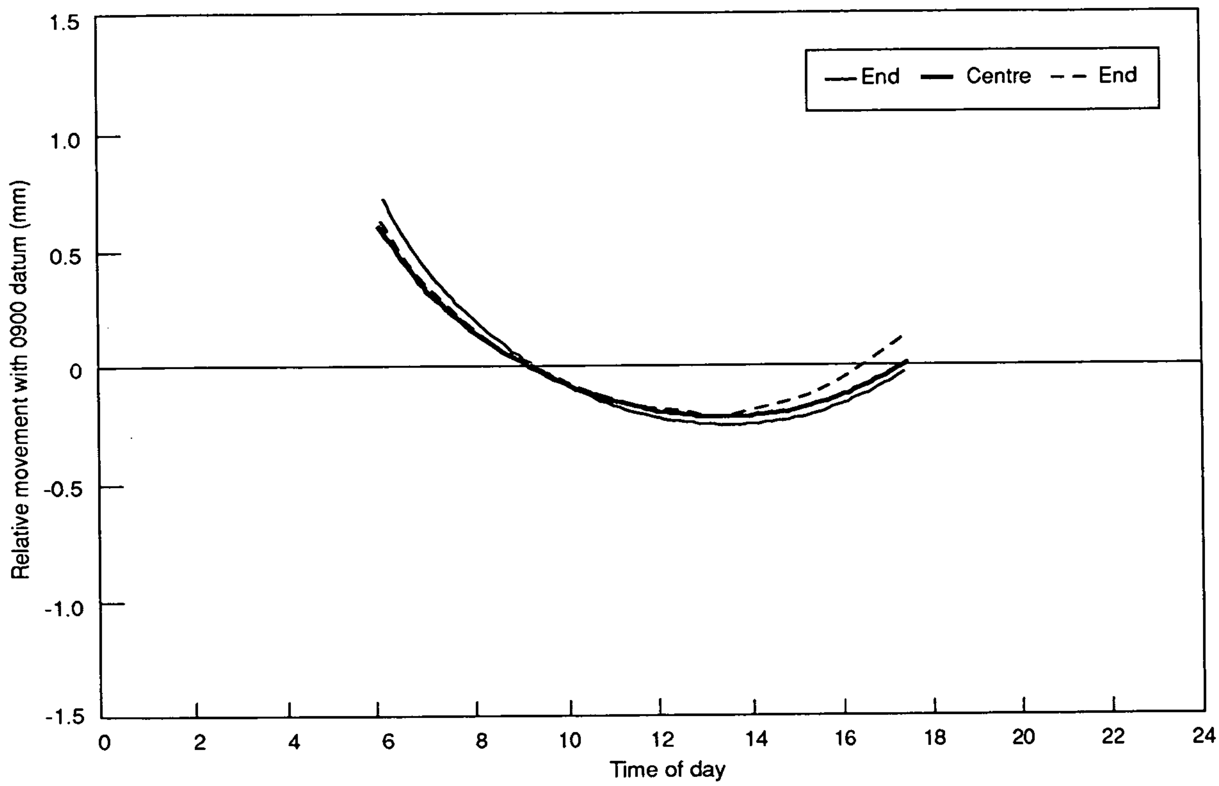


Fig. 14 Transverse warping of slab C1-97

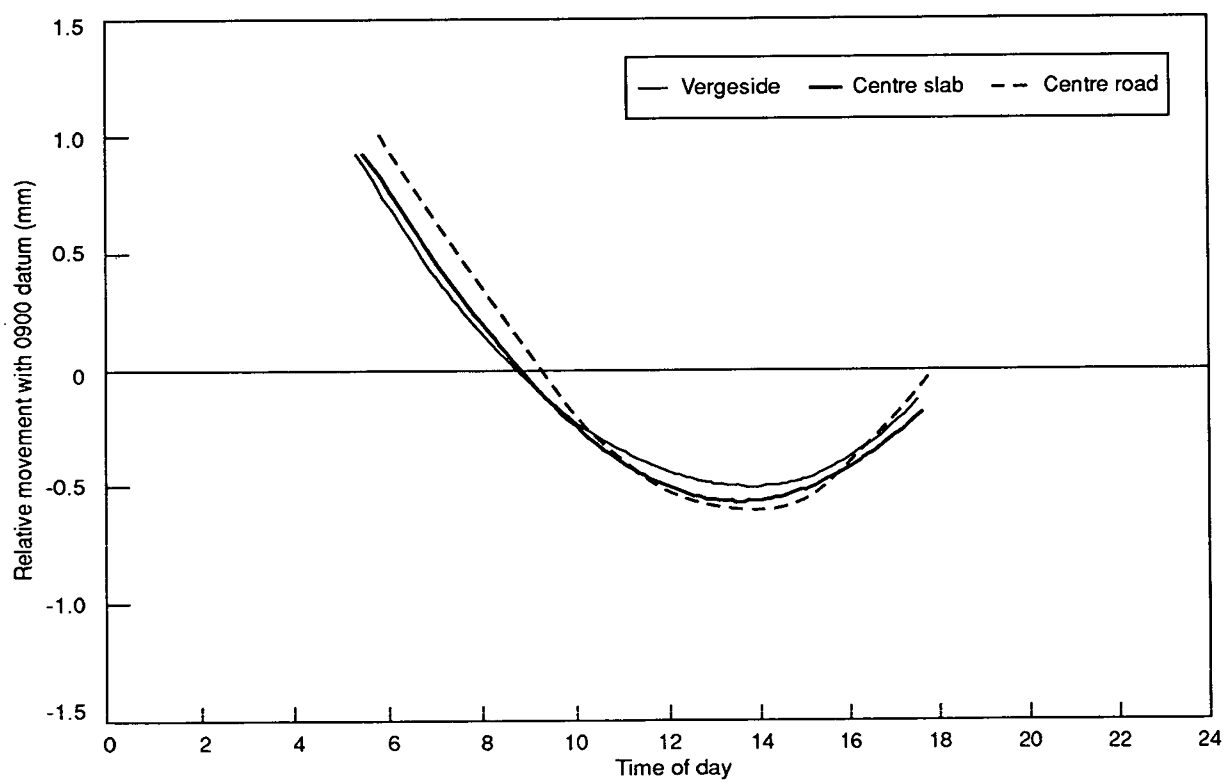


Fig. 15 Longitudinal warping of slab C1-97

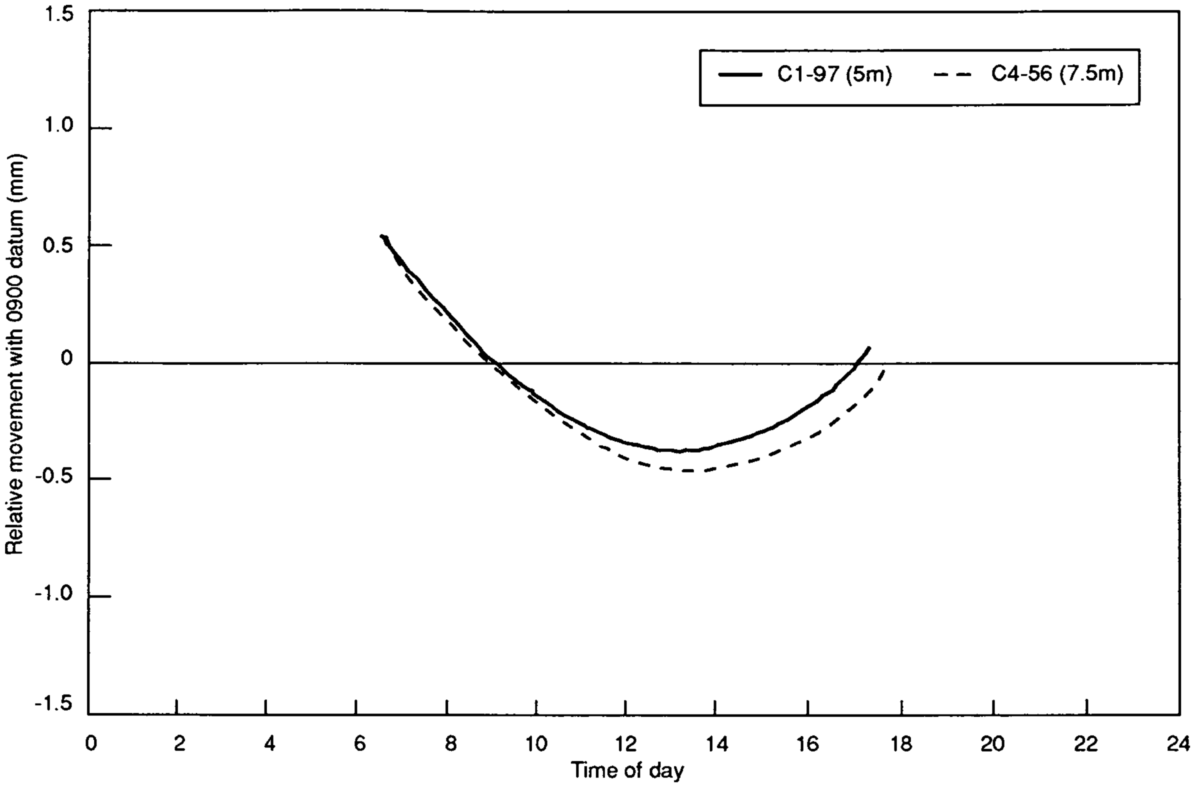


Fig. 16 Transverse warping of slabs of different lengths

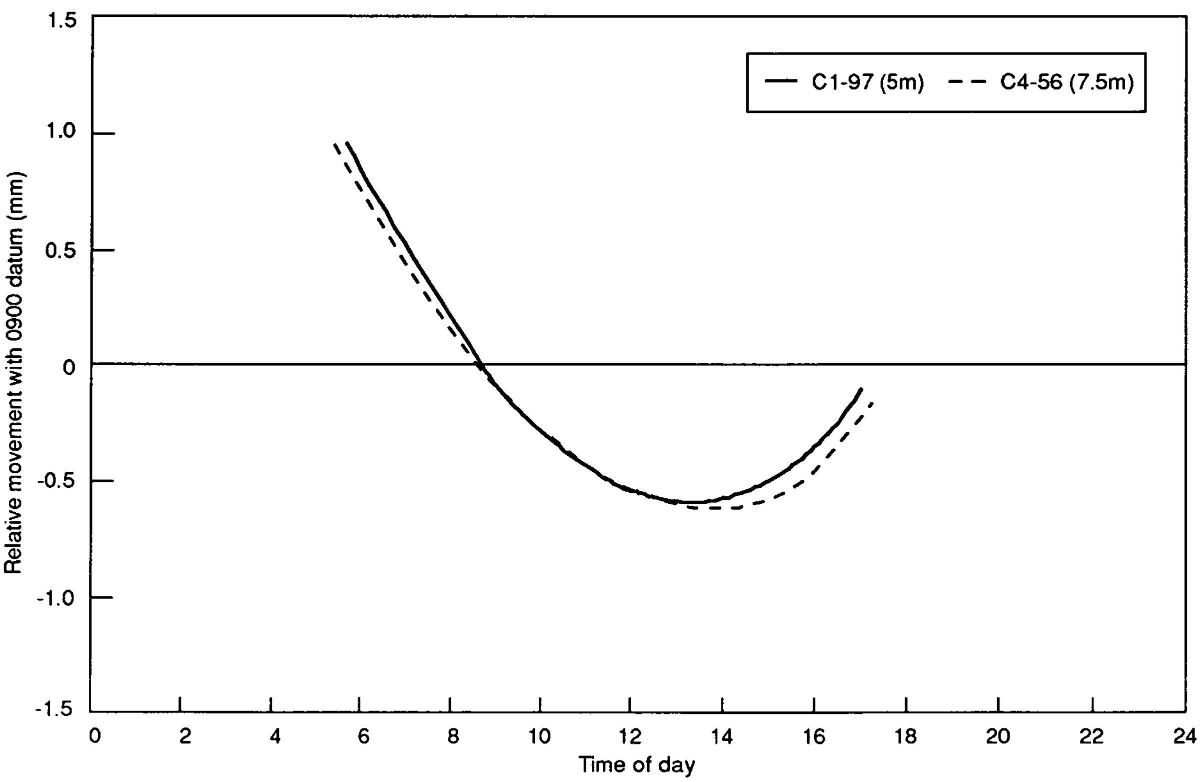


Fig. 17 Longitudinal warping of slabs of different lengths

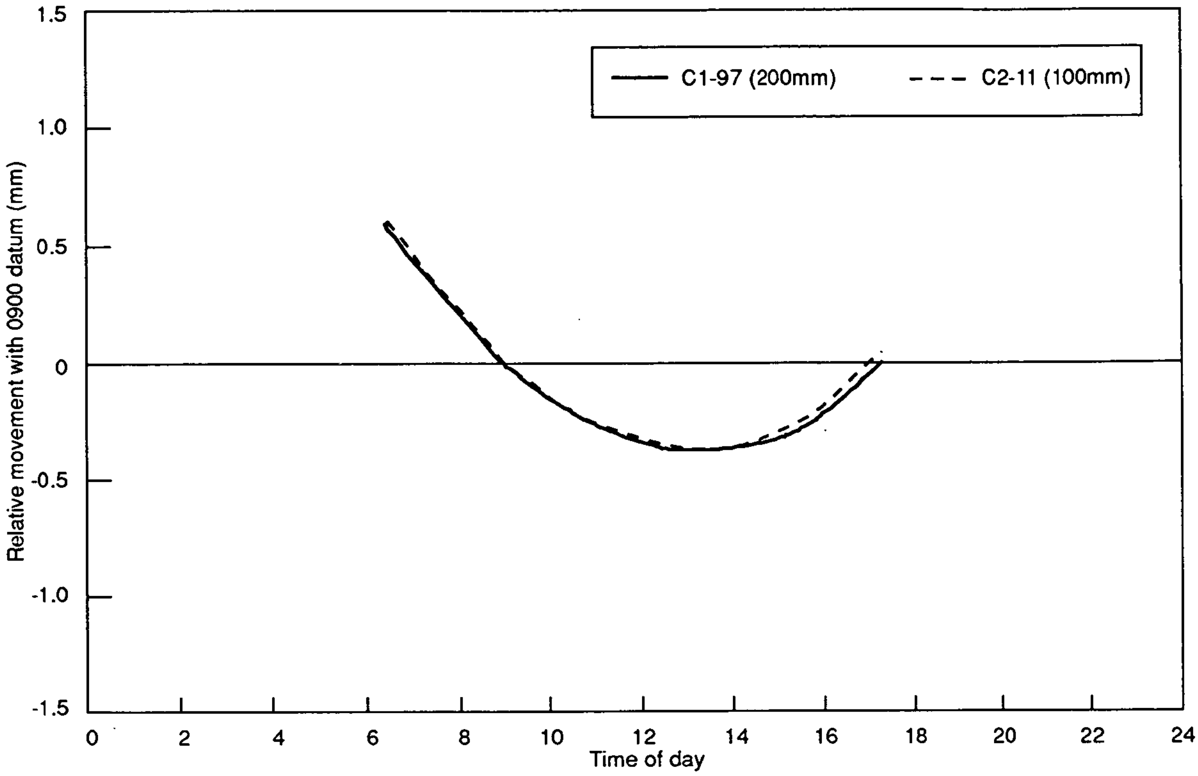


Fig. 18 Transverse warping of slabs of different thickness

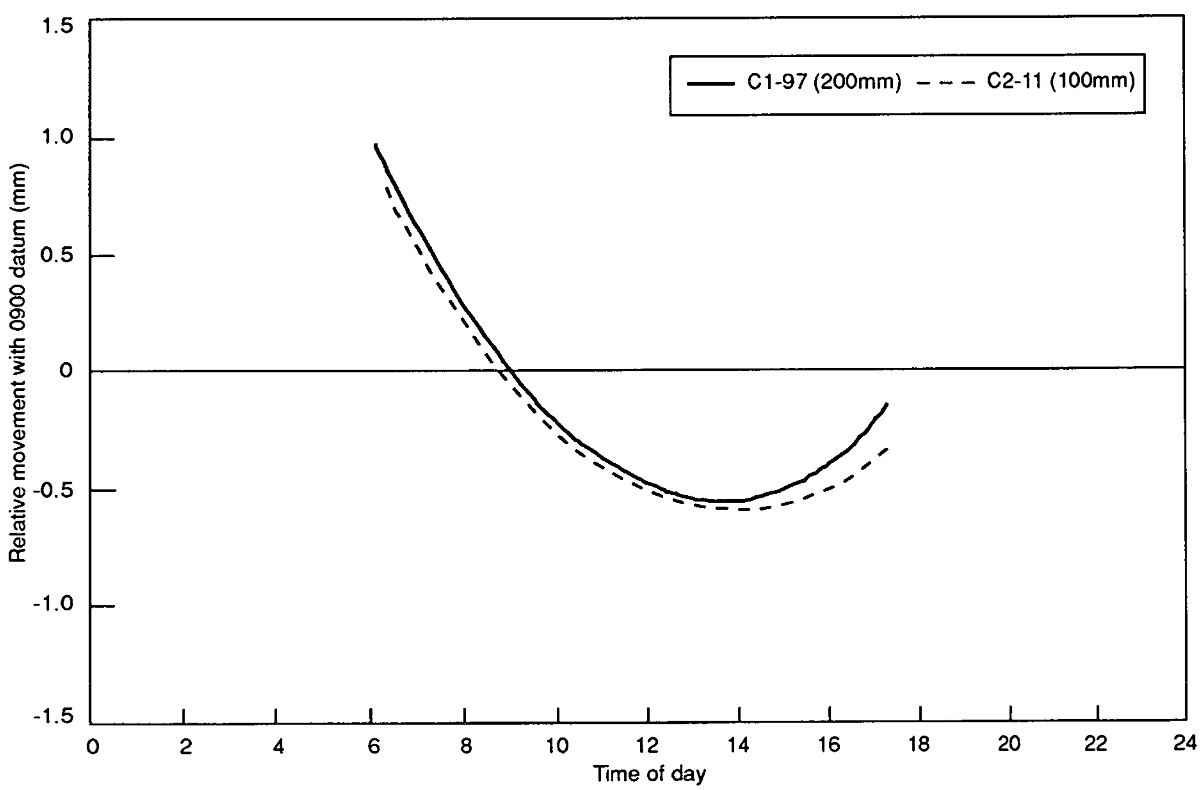


Fig. 19 Longitudinal warping of slabs of different thickness

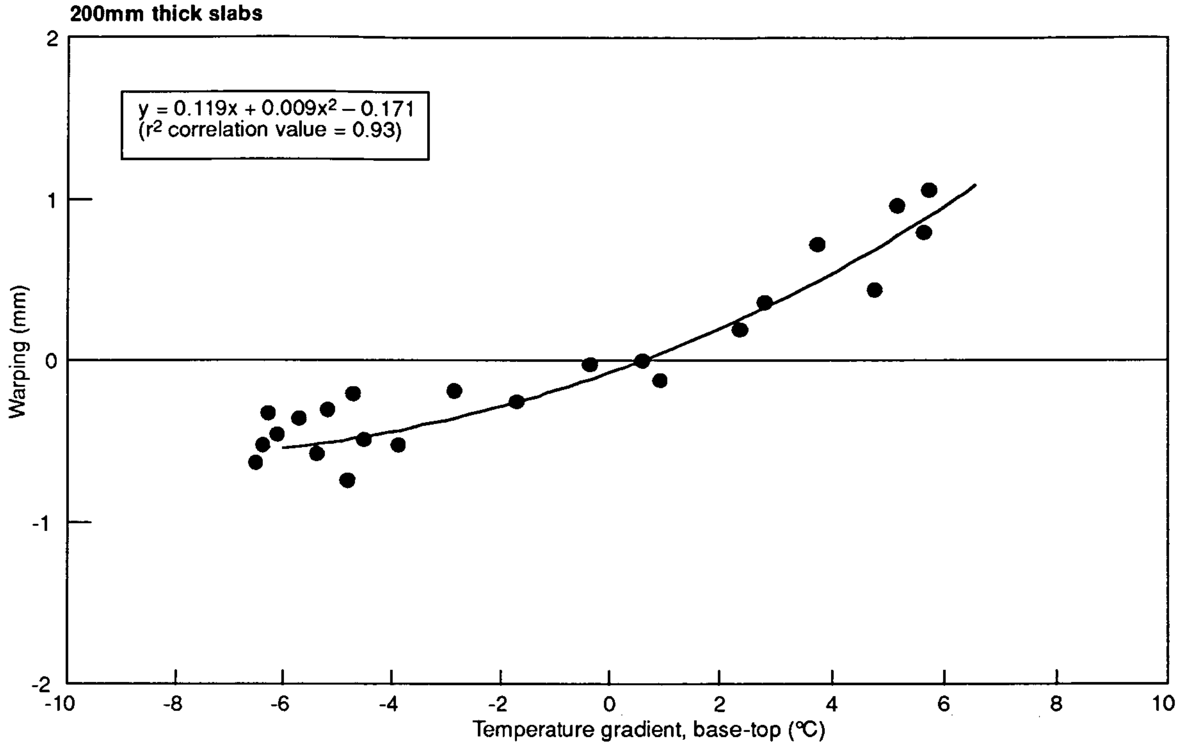


Fig. 20 Longitudinal warping plotted against temperature gradient (daytime)

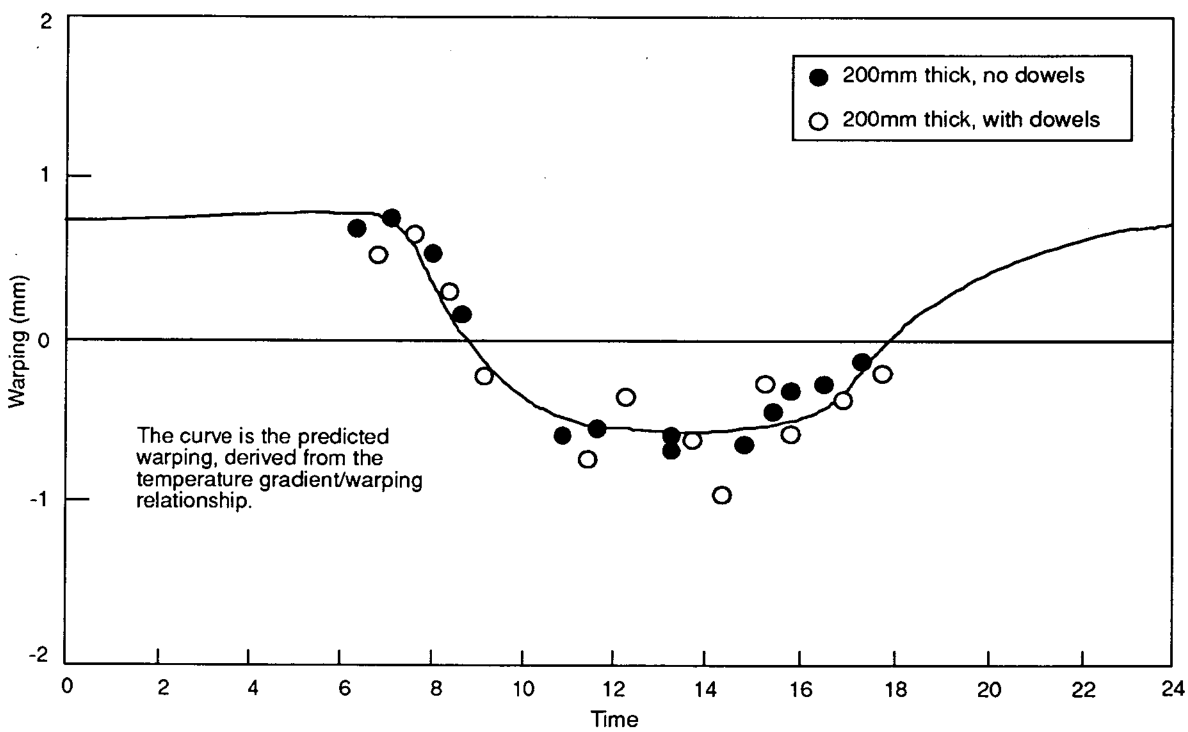


Fig. 21 Longitudinal warping plotted against time (24 hours)

the two slabs, one dowelled at the ends, the other undowelled. Calculations showed the relationship to be similar for slabs of less thickness. Figure 22 shows the same curve with measured results on slabs 100mm, and 140mm thick. Clearly, variations in the weather will affect the magnitude of the warping by causing the temperature differential between the top and base of the concrete slabs to vary. This in turn also affects the deflection measurements as the slabs rock under load.

4.7.3 Effects of warping

The general conclusion from this analysis is that concrete road slabs deform over a 24 hour period, tending to leave a void beneath the centre in the day time and a void under the edges at night. Theoretical analysis of the effects of temperature changes on concrete slabs (Croney, 1991; Road Research Laboratory, 1956) concludes that slab thickness, length, sub-base and other factors influence the movements and internal stresses. The measurements recorded here, taken under clear sky conditions in winter at a latitude of 20° south, show little difference in the movement of slabs with thicknesses varying from 100mm to 200mm and lengths of 5m and 7.5m. This is due largely to the effect of the concrete mass, which acts against the warping tendency, increasing the internal stress.

Day-time lack of support at the centre causes bending stress in the slabs due to self weight, which is additional to the bending stress due to applied loads, and leads to transverse cracking followed by longitudinal cracking. Night-time lack of support at the ends and sides of the slabs promotes rocking under load, joint wear, water penetration and hence pumping out of fines from the sub-base, which increases the rocking effect. Lack of edge support makes corner cracking more likely, as well as transverse cracking which starts at the outer edges.

Stresses caused in this manner cannot be deduced by simple calculation because there is also, in practice, a deformation of the sub-base. Day-time hogging concentrates all the weight of the slabs and applied load around the edges, which tend to be wetter than the centre. This causes some deformation. Night time dishing causes rocking and pumping, which also tend to erode the sub-base under the edges of the slabs. The effect of this can be hogging of the sub-base under each slab and this results in a tendency towards reasonably uniform support to the concrete slab during the day and more severe lack of support at the sides and ends at night. Under these conditions shear loads at joints are increased further, accelerating wear and sealant failure, with subsequent further deterioration of sub-bases leading to slab rocking, even during the day when the slabs are hogged in shape. The degree of sub-base erosion is dependent on the type of material, the age of construction and the amount of water that has been permitted to enter the sub-base.

Although they are not as extreme as under tropical desert conditions, the diurnal temperature variations illustrated above are more severe than those found in most coun-

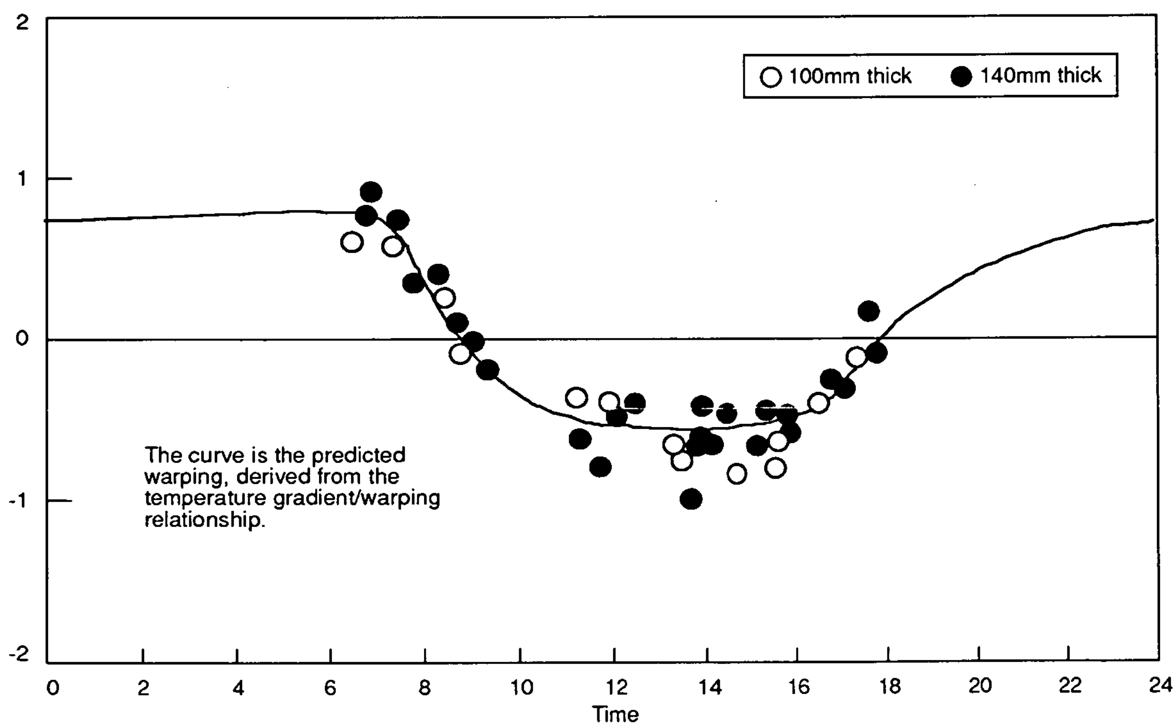


Fig. 22 Longitudinal warping plotted against time (24 hours)

tries, and indeed in Zimbabwe during most of the year. However, a surface temperature variation in the order of 17°C is shown to cause warping, i.e. a change between day and night shape in excess of 1mm, in road slabs made with granite aggregate. Limited variations in slab length and thickness and the presence of dowels at the joints had no measurable effect on this movement.

Warping is not the only cause of joint deterioration and sub-base erosion on concrete pavements, but the relationship between the diurnal temperature variation at the slab surface and slab rocking makes jointed concrete pavements less suitable in areas where these temperature variations are high.

4.7.4 Slab length

Transverse cracking of jointed slabs occurs when the total bending stress due to applied loads, self weight and manner of support exceeds the flexural fatigue strength of the concrete. Clearly the length of the slab has an effect on the stress, both in the day-time hogging shape and in the night-time dishing shape. The type of aggregate used in the concrete affects the slab design length because different aggregates affect the coefficient of linear expansion. Most design manuals take this into account (CPCA, 1984; AASHTO, 1974; RRL, 1970). The degree of warping too is dependent on slab length, but also on the diurnal temperature range within the slabs and therefore this should also be a factor to be considered when specifying slab length.

4.8 SHOULDERS

During construction of the shoulders the natural gravel was distributed on each side of the concrete pavement, graded and mixed by hand with 5% cement. It was compacted using pedestrian operated vibrating rollers and was then sealed with a single bitumenised surface dressing. The shoulder material soon shrank leaving a gap of at least 3mm against the concrete pavement and a step of 10mm or more from the concrete down to the shoulder. A fillet of bitumenised material was applied but the gap soon opened again. Unstabilised, well graded gravel shoulder material would have been more successful because:

a) it would not have shrunk

b) it would have been plastic enough to maintain contact against the concrete.

On concrete pavements, tied concrete shoulders are more successful than shoulders built with unbound aggregates. More cement is required but the bitumenised surface dressing and the cement costs are similar. Furthermore, a reduction in concrete pavement thickness ranging from 10% to 15% is permitted when tied shoulders at least one metre wide are specified (DTp Standard, 1987; CPCA, 1984).

5. MAINTENANCE

Over the first six and a half years of use, maintenance on the test sections has been restricted to:

a) attempts to seal the gap between the concrete and the stabilised shoulders by the application of a bituminous fillet and

b) replacement of transverse joint seals that failed in early life.

All the joint seals now require replacement. All the remaining sealant should be removed, the grooves cleaned with an abrasive and resealed. There is a variety of sealant material available but a hot applied sealant to BS 2499 (1973) is recommended as a first choice.

The gap between the concrete and the stabilised shoulders is more difficult to seal. However, the same sealant as used in the transverse joints would be more successful than the bitumen used to-date. Furthermore, the sealant would suffer less strain if a conventional groove were cut in the shoulder material against the concrete to provide a joint width in the order of 15mm to 20mm. This would enable the surfaces to be cleaned before applying the sealant as well as reducing the strain in the material during temperature induced movements and slab rocking. However, since it is not known whether the sealant would adhere to the shoulder material, a trial length of about 100 metres should be tried first.

The DTp Advice Note HA 35/87 (1987) states, "During the life of the road, some replacement of the concrete slabs will be required in addition to the resealing of joints and any arris and thin bonded repairs that may be required."

The following maintenance is recommended to take place in 1993, after 7 years and approximately 250,000 ESAs of traffic:

i) Replacement of seven entire slabs

ii) Full depth repair or replacement of sixteen further slabs

iii) Thin bonded arris or corner repairs at twenty five locations.

The above repairs should be carried out according to the DTp/C&CA publication by Mildenhall (1986). They are discussed in detail in PR/OSC/025/93 and the locations are marked on Figures A1 to A4.

6. SUMMARY AND RECOMMENDATIONS

1) The contractor proved that good quality concrete can be produced using relatively small items of equipment that were either old or locally-made. The combination of vibrating pokers and two light vibrating screeds was very effective in compacting a stiff concrete mix (generally 15-30mm slump) throughout depths ranging from 80mm to 237mm.

2) A daily temperature variation at the top of the concrete pavement of 17°C caused the slabs to warp by more than 1mm (measured as a change in shape of the slab centre relative to the slab edges). This warping movement was not measurably affected by slab length (5m to 7.5m) or by slab thickness (100mm to 200mm) or the use of dowels at transverse joints.

3) The weight of the concrete acting over the unsupported areas of a slab, which is warped to the extent that it is not in full contact with the sub-base, acts against the warping force and so stresses the material. This bending stress is proportional to the length of unsupported slab and is additional to that resulting from traffic loads. Hence diurnal temperature variations should be considered when designing slab lengths. Until more data become available from other sites, an interim recommendation restricting unreinforced slab lengths to 4.5m (5m using limestone aggregate) would be advisable where top surface temperatures vary by 15°C or more daily.

4) Stabilised gravel shoulders are not recommended for use with concrete pavements because of the difficulty in sealing the gap between the pavement and the shoulder caused by shrinkage of the stabilised material.

5) Deflections measured on jointed concrete pavements are affected by the warping of the slabs, which in turn is influenced by weather conditions. Variations in weather conditions have masked any trend in the deflection measurements made over five years.

6) The electronic deflection logger that was developed as part of this programme has proved to be a useful and reliable tool for the measurement of vertical deflections on slabs and horizontal strains at joints and cracks on this and other concrete pavements.

7) A significant amount of fine material has been eroded from the subgrade directly below the slabs on sections C3 and C4, and to a lesser extent from the sub-base material on sections C1 and C2. It is concluded that concrete pavements require a non-erodible sub-base, as currently specified in the DTp Standard (1987) even under these conditions of relatively light traffic and mostly dry climate i.e. 50,000 ESAs and 670mm of rain per year.

8) The transverse joints on all the trial sections should now be resealed with an elastomeric material conforming to the standards listed in the DTp/C&CA publication by Mildenhall (1986). Some slab replacement and repair is also required.

9) The small amount of faulting that has occurred is not detectable by a change of roughness as measured by the TRL Profile Beam.

10) Skidding resistance on all trial sections is well above the recommended minimum, although there is a small reduction in the skidding resistance value measured in recent years. There has been no change in the measurements of texture depth. The randomised spacing of the texture grooves is believed to reduce tyre noise.

11) Transverse joints cut parallel to texture grooves permit very little water to penetrate to the sub-base, even when the seals are damaged. This advantage would clearly be reduced on longitudinal gradients, so both joints and texture grooves should be skewed to follow the maximum slope.

12) Local deviations in the finished levels of sub-base on Sections C1 and C2 (and subgrade on Sections C3 and C4) and accurate finishing of the top surface of the concrete slabs led to wide variations in the thickness of the concrete on all four trial sections. These variations together with slab and sub-base strength measurements will be extremely useful, when enough deterioration has occurred, for conclusions to be drawn concerning thickness design of concrete pavements under similar conditions of traffic, climate and materials. To-date, cracking due to traffic is confined to two locations where the concrete is only 110mm to 115mm thick, the damage most likely being caused by the passage of specific vehicles with axles weighing about 14 tonnes in 1987 and 1988.

7. CONCLUSIONS

The project has been managed from TRL and has included annual site visits with excellent cooperation from the Research Branch and the Central Laboratory of the Zimbabwe Ministry of Transport. This collaborative method of working has proved to be effective and economical in the use of resources.

It has been demonstrated that concrete pavements, constructed using only basic machinery, are viable under the conditions of traffic, climate and materials availability in Central Africa.

Much of the data and recommendations summarised above will be applicable to other regions also, and will be incorporated in a manual on the design and construction of concrete roads in the Overseas Road Note series.

8. ACKNOWLEDGEMENTS

This work has been part of a continuous programme of collaborative research undertaken by the Zimbabwe Ministry of Transport and the Overseas Centre of the UK Transport Research Laboratory. Thanks are given to Mr R Mitchell, Director of State Roads, and all the engineers and technicians in Zimbabwe who have made contributions during the construction and subsequent condition assessments.

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APPENDIX A: FIGURES A1 TO A6, TABLES A1 TO A17

FIGURE A1

SECTION C1

VISUAL CONDITION SURVEY

DATE: 7-7-92

\ =CRACK. \ =CONSTRUCTION JOINT. JOINT CONDITION: G-GOOD, F-FAIR, B-BAD

RECOMMENDED REPAIRS: SR - SLAB REPLACEMENT, FD - FULL DEPTH REPAIR, TB - THIN BONDED REPAIR

1 B	3 B	5 B	7 F	9 B	11 B	13 TB B	15 B	17 B	TB 19 B
2 B	4 B	6 B	:01 8 B	10 F	12 F	14 B	16 B	18 B	20 B

17+480

21 B	23 B	25 B	27 B	29 B	31 B	33 B	35 B	37 B	39 B
22 B	24 B	26 B	28 B	30 B	32 B	34 B	36 B	38 B	40 B
41 B	43 B	45 B	47 B	49 B	51 B	53 B	55 B	57 F	59 F
42 B	44 B	46 B	48 B	50 B	52 B	54 B	56 B	58 F	60 B
61 B	63 B	65 F	67 B	FD 69 B	71 B	73 B	75 G	77 B	79 B
62 B	64 B	66 F	68 B	70 B	72 B	74 B	76 B	78 B	80 B
81 B	83 F	85 B	87 B	89 B	91 B	93 B	95 B	97 B	99 B
82 B	84 B	86 B	88 B	90 B	92 B	94 B	96 B	98 B	100 B
101 B	103 B	105 B	107 B	109 B	111 B	113 B	115 B	117 B	119 B
102 B	104 F	106 B	108 B	110 B	112 B	114 B	116 B	118 B	120 B
121 B	123 B	125 F	127 F	SOUTH SIDE	NOTES	SUPERELEVATION WHOLE SECTION - NORTH SIDE			
122 B	124 B	126 F	128 F	NORTH SIDE		HIGHER. EXTENSIVE SHOULDER SHRINKAGE			

17+800

SLIGHT MUD STAINING ON BOTH SHOULDERS.						CRACKING - TRANSVERSE CRACKS SEALED,			
						UNSEALED GAP BETWEEN SHOULDER & CONCRETE.			

FIGURE A2

SECTION C2

VISUAL CONDITION SURVEY

DATE: 7-7-92

\ =CRACK. \ =CONSTRUCTION JOINT. JOINT CONDITION: G - GOOD, F - FAIR, B - BAD

RECOMMENDED REPAIRS: SR - SLAB REPLACEMENT, FD - FULL DEPTH REPAIR, TB - THIN BONDED REPAIR

1 F	3 F	5 F	7 F	9 F	11 G 1.0	13 G	15 F	17 F	19 F
2 F	4 B	6 F	8 F	10 F	12 F	14 F 2.3	16 SR F	SR 18	2.0 F 20 B

17+800

21 F	23 B	25 F	27 F	29 F	31 F	33 B	35 F	37 B	39 B
22 B	24 FD G	0.5 26 F	28 F	30 1.4 TB F	32 F	34 F	36 F	38 F	40 1.5 B

41 B	43 FD F	45 F	47 F	49 B	51 F	53 F	55 F	57 F	59 F
42 F FD	44 F	46 F 0.4 FD	48 F	50 F	0.5 52 F	54 F	56 B	1.0 58 B	60 B

61 F FD	63 F	65 B	67 B	69 B	71 B	73 F	75 F TB	77 F	79 F
62 F	64 B	66 B	68 F	70 B	72 F	74 F	76 F	78 F	80 F

81 F	83 F	85 B	87 B	89 B	91 F	93 B	95 B	97 B	99 F
82 F	84 F	86 F	88 B	0.3 FD 90 B	92 F	94 F	96 F	98 B	100 B

← SQUARE JOINTS SKEW JOINTS →

101 F TB 1.0	103 F	105 B	107 B	109 B	111 B	113 B	115 B	117 B	119 B
102 F	104 B	106 B	108 B	110 B	112 B	114 FD B	116 B	118 FD B	120 B

121 B	123 SR B	FD 125 B	127 B	129 B	SOUTH SIDE	NOTES	SUPERELEVATION WHOLE	SECTION-
122 B	124 B	126 B	128 B	130 B	NORTH SIDE	NORTH SIDE HIGHER.	SHOULDER	SHRINKAGE

18+120

SLIGHT MUD	STAINING ON	SOUTH SHOULDER				CRACKING -	TRANSVERSE	ON NORTH	SIDE,
					ON SOUTH SIDE	UNSEALED	GAP BETWEEN	SHOULDER &	CONCRETE

FIGURE A3

SECTION C3

VISUAL CONDITION SURVEY

DATE: 7-7-92

\ =CRACK. \ =CONSTRUCTION JOINT. JOINT CONDITION: G - GOOD, F - FAIR, B - BAD

RECOMMENDED REPAIRS: SR - SLAB REPLACEMENT, FD - FULL DEPTH REPAIR, TB - THIN BONDED REPAIR

1 B	3 B	5 B	0.4 FD 7 B	9 B	11 B	13 0.3 FD	B 15 B	17 B	19 B
2 B	4 B	6 B	0.2 8 B	10 B	12 B	14 0.2	B 16 B	18 B	20 B

18+580

21 B	0.4 FD 23 B	25 B	27 B	29 B	31 B	33	B 35 B	37 B	39 F
22 B	0.2 24 B	26 B	28 F	30 B	TB 32 F	34 0.8 TB	F 36 B	38 B	40 F

41 F	43 B	45 B	47 B	49 B	51	B 53 B	55 B	57	B 59 B
TB 42 B	44 0.7 TB	B 46 F	48 B	50 B	52 FD	G 54 F	56 F	58	B 60 B

61 B	63 B	65 B	67 B	69 B	71 B	FD 73 B	75 B	77 F	79 F
62 B	64 B	66 B	TB 68 F	70 B	72 B	74 B	76 F	78 F	80 F

81 B	83	F 85 F	87 F	89 B	91 B	93 F	95 B	97 B	99 B
82 F	84 2.0 FD	F 86 G	88 F	90 F	92 F	94 F	96 1.2 G	98 F	100 F

101 B	103 B	0.2 105 FD	G 107 B	109 B	FD 111 B	113 F	115 B	117 B	119 B
102 F	104 F	106	F 108 F	110 B	112 B	114 F	116 B	118 B	120 B

121 B	123 B	125 B	127 B	129 B	131 B	133 B	135 B	137 B	139 B
122 B	124 B	126 B	128 B	130 B	132 B	134 B	136 B	0.5 TB 138 B	140 B

141 B	143 B	145 B	147 B	149 B	151	SOUTH SIDE UNSEALED GAP BETWEEN SHOULDER &
142 B	144 B	146 B	148 B	150 B	152	NORTH SIDE CONCRETE WITH MUD STAINING

18+960

NOTES

FIGURE A4

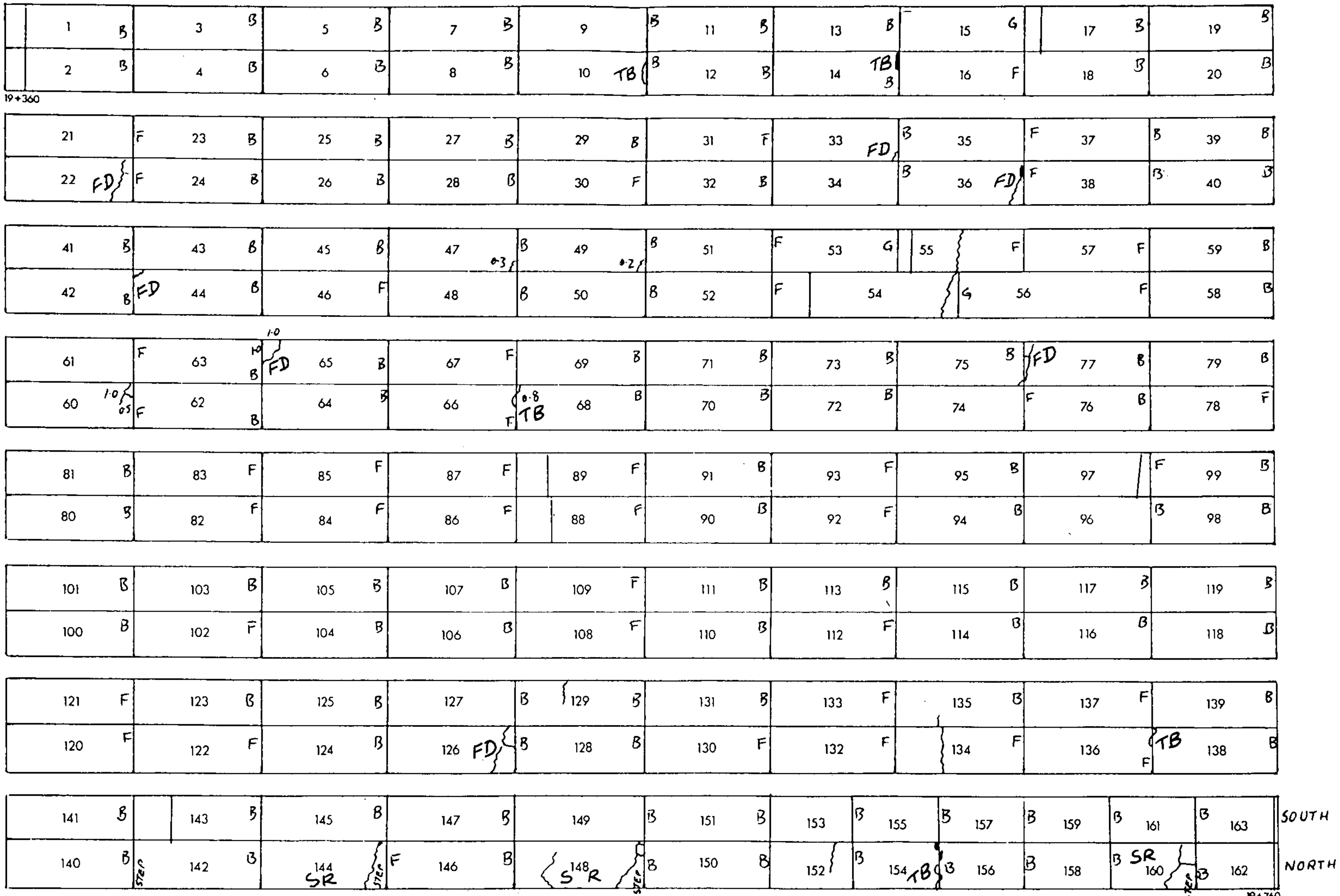
SECTION C4

VISUAL CONDITION SURVEY

DATE: 7-7-92

\ = CRACK. \ = CONSTRUCTION JOINT. JOINT CONDITION: G - GOOD, F - FAIR, B - BAD

RECOMMENDED REPAIRS: SR - SLAB REPLACEMENT. FD - FULL DEPTH REPAIR. TB - THIN BONDED REPAIR



NOTES. SUPERELEVATION FROM SLAB 80 SOUTH SIDE HIGH. PRONOUNCED MUD STAINING NORTH SIDE. SOME SHOULDER SHRINKAGE BOTH SIDES.

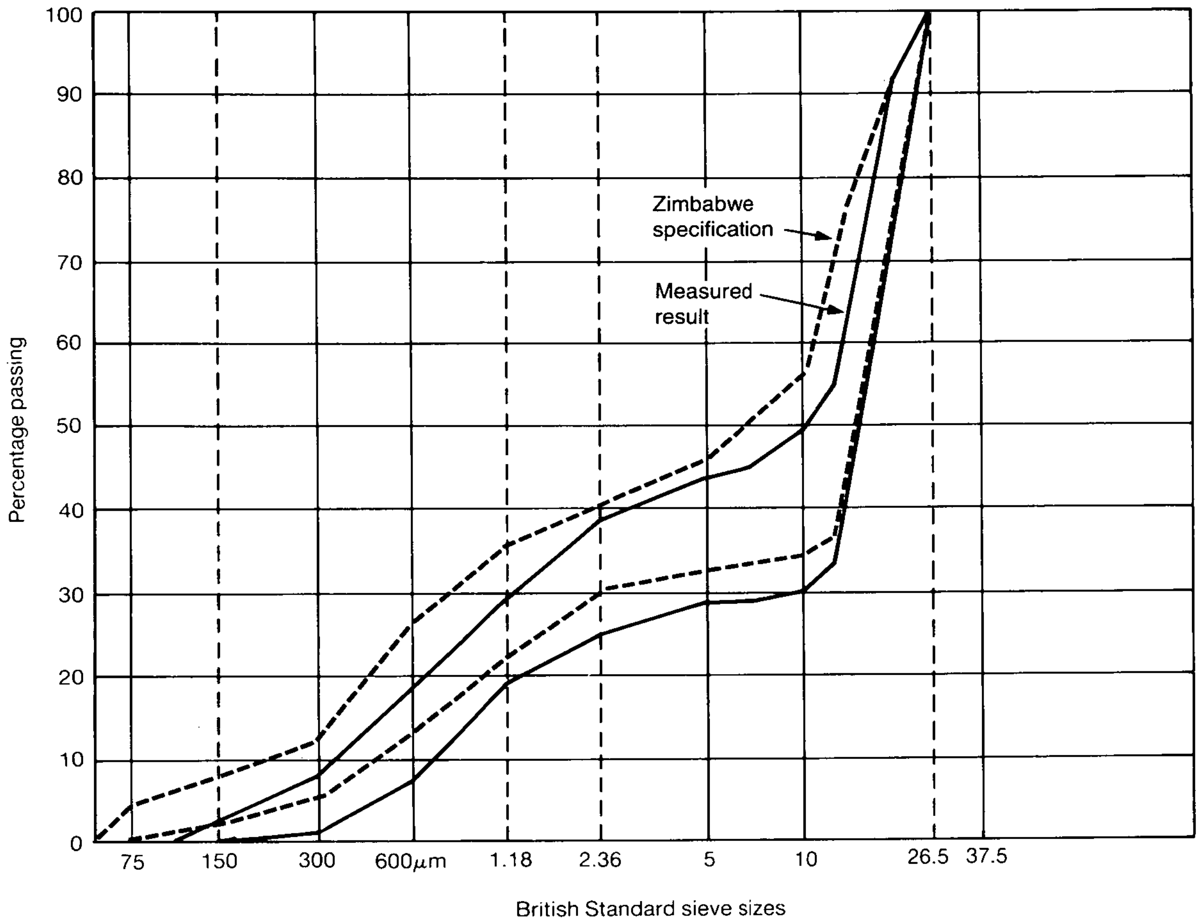


Fig. A5 Wet sieving results – concrete aggregates

Run No 36 Date 13/10/92 Time 16:34:31 Sample Time 10 s Section 11 Chainage 111

Values in mm

VERTICAL 1 :

0.00	0.00	0.01	0.00	0.00	0.01	0.01	0.01	0.01	0.02
0.02	0.03	0.04	0.06	0.08	0.10	0.13	0.17	0.18	0.19
0.15	0.12	0.07	0.02	-0.02	-0.07	-0.11	-0.15	-0.18	-0.20
-0.22	-0.24	-0.25	-0.26	-0.27	-0.27	-0.27	-0.27	-0.27	-0.28
-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.26	-0.26	-0.26

VERTICAL 2 :

0.00	-0.00	-0.00	-0.00	-0.01	-0.00	-0.00	0.00	0.00	0.01
0.01	0.02	0.03	0.04	0.06	0.08	0.10	0.12	0.12	0.13
0.08	0.04	-0.00	-0.05	-0.10	-0.14	-0.18	-0.21	-0.24	-0.26
-0.28	-0.31	-0.30	-0.31	-0.32	-0.33	-0.32	-0.32	-0.32	-0.32
-0.32	-0.33	-0.32	-0.32	-0.31	-0.31	-0.32	-0.31	-0.31	-0.31

VERTICAL 1 SUMMARY :

INITIAL VALUE :	0.00
MAXIMUM VALUE :	0.19
MINIMUM VALUE :	-0.28
FINAL VALUE :	-0.26
TRANSIENT DEFLECTION :	0.32

VERTICAL 2 SUMMARY :

INITIAL VALUE :	0.00
MAXIMUM VALUE :	0.13
MINIMUM VALUE :	-0.33
FINAL VALUE :	-0.31
TRANSIENT DEFLECTION :	0.29

HORIZONTAL :

0.00	0.00	0.00	0.00	0.00	-0.00	-0.00	-0.00	0.00	-0.00
-0.00	-0.00	-0.00	-0.01	-0.01	-0.02	-0.02	-0.03	-0.03	-0.03
-0.02	-0.01	-0.00	0.00	0.01	0.02	0.02	0.02	0.02	0.02
0.02	0.02	0.02	0.01	0.01	0.01	0.01	0.01	0.01	0.01
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

HORIZONTAL SUMMARY :

INITIAL VALUE :	0.00
MAXIMUM VALUE :	0.02
MINIMUM VALUE :	-0.03
FINAL VALUE :	0.00
TRANSIENT DEFLECTION :	0.02

RELATIVE [V1-V2] SUMMARY :

INITIAL VALUE :	0.00
MAXIMUM VALUE :	0.07
MINIMUM VALUE :	0.00
FINAL VALUE :	0.04

V1 ******   V2 ++++++   H ......   V1-V2 ######   Plot Magnification Factor = 4

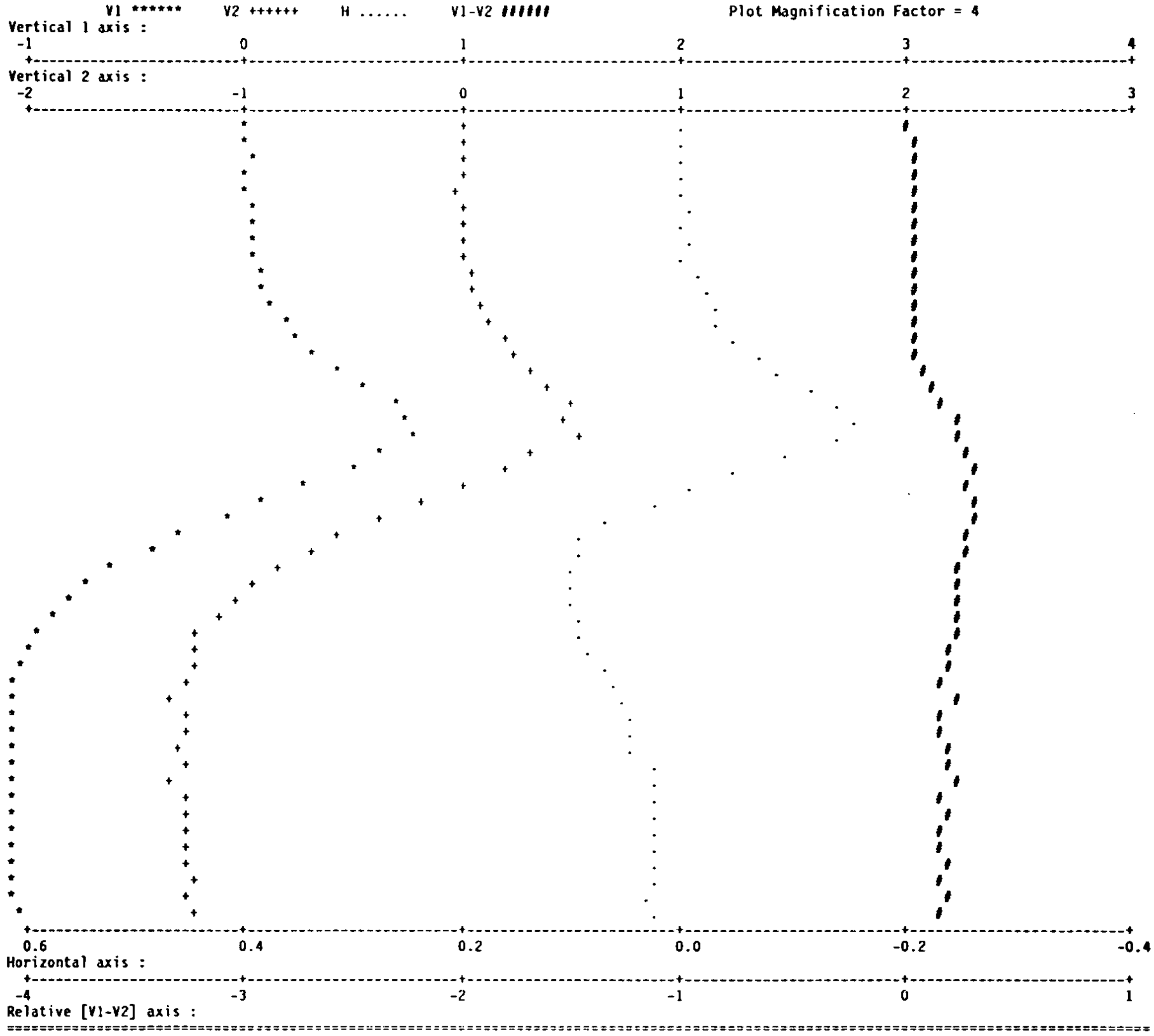


Fig. A6 Typical deflection logger print and plot

TABLE A1

Two-way axle load measurements

Axle Load (tonnes)	1985	1987	1988	1989	1990	1991	1992
				No. of axles from 7 day count			
<2	394	133	140	166	134	400	240
<3	669	343	357	304	286	447	515
<4	461	308	252	222	288	393	513
<5	421	322	294	250	336	453	429
<6	408	238	217	355	457	482	695
<7	207	133	28	166	431	311	601
<8	150	63	7	102	286	252	341
<9	68	0	21	63	195	107	177
<10	15	0	1	22	80	33	81
<11	3	7	0	7	9	6	15
<12	0	0	0	1	3	2	3
<13	0	0	0	0	0	0	2
<14	1	0	0	0	0	0	0
14+	0	0	0	0	0	5	0
Damaging factor (ESAs/week)	569	262	156	491	831	896	1070
Total Damaging factor (ESAs/year)	29,600	26,100*	20,600*	25,500	43,200	46,600	55,600
Total ESAs 1987-92:				217,600			

ESA = Equivalent Standard Axle.
* 1987 and 1988 each include 12,500 ESAs as the estimated effect of aggregate lorries.
July 1987 and 1988 were estimated from several 8 hour counts.

TABLE A2

Two way traffic counts (vehicles per day)

	1986	1987	1988	1989	1990	1991	1992
All vehicles:	243	365	221	297	430	425	567
Buses and HGVs:	82 (34%)	128 (35%)	76 (34%)	83 (28%)	130 (30%)	126 (33%)	166 (29%)

TABLE A3

Results of beam test results at TRL - concrete mix trials

Beam No.	28 day Flexural Strength (MPa)	Density (Kg/m^3)	Dynamic E (N/mm^2 x10^3)	Equivalent cube strength (MPa)	Notes
51	4.53	2400	41.0	42.0	
52	4.14	2375	40.5	43.5	
53	3.91	2420	42.0	43.5	
54	3.62	2411	41.0	42.0	* slow loading rate
56	4.04	2406	40.0	42.0	
59	3.95	2427	41.5	41.0	
62	3.76	2412	41.5	39.0	* large stone in fracture
64	3.92	2419	41.0	38.0	
65	3.87	2422	41.5	40.5	
Average:	3.97	2410	41.1	41.3	

TABLE A4

Results of beam tests in Zimbabwe - concrete mix trials.

Beam No.	28 day Flexural Strength (MPa)	Equivalent cube strength (MPa)
48	3.54	39.0
49	4.04	35.0
50	4.05	39.0
55	4.16	35.0
57	4.05	39.0
58	4.31	38.0
60	3.97	35.0
61	Beam Damaged	-
63	3.88	39.6
Average:	4.00	37.5

TABLE A5

Sub-base and subgrade material properties

Sub-base Standard	% Passing (sieve sizes in mm)								
Triaxial class 3.3	26.5	19	9.5	4.75	2.36	425μ	75μ	Plasticity Index	Plasticity Product
% passing	89-100	75-100	53-100	37-75	26-75	11-42	5-27	mean = 10 worst = 12	mean = 150 worst = 200

Sub-base Test Results

Section	Chainage	26.5	19	9.5	4.75	2.36	1.18	600μ	300μ	150μ	75μ	Rejection Index	Liquid Limit	Plasticity Index
		% Passing (sieve sizes in mm)												
C1	17.520 C	96	92	83	73	61	52	45	37	29	25	5	27	11
Sub-base	17.580 L	97	95	88	80	63	48	41	35	30	25	5	21	5
	17.640 C	93	91	77	64	54	47	42	34	25	20	4	22	6
	17.700 R	95	92	80	66	55	47	41	34	25	21	nil	23	11
	17.760 L	98	93	81	65	54	47	41	33	24	20	nil	23	9
C2	17.840 C	97	93	82	69	55	48	41	34	25	22	2	23	8
Sub-base	17.900 L	99	92	83	71	62	55	44	32	28	23	1	23	13
	17.960 C	94	86	75	63	53	48	42	34	25	20	nil	22	9
	18.020 R	98	96	90	82	71	57	49	42	34	32	4	29	4
	18.080 L	96	92	79	65	55	48	42	32	23	19	6	22	10

Subgrade Test Results

Section	Chainage	26.5	19	9.5	4.75	2.36	1.18	600μ	300μ	150μ	75μ	Rejection Index	Liquid Limit	Plasticity Index
		% Passing (sieve sizes in mm)												
C1	17.520 C	97	95	87	77	61	49	41	34	28	22	nil	25	11
Subgrade	17.580 L	95	93	82	71	58	51	44	36	27	22	4	29	8
	17.640 C	96	93	84	74	55	42	36	31	26	23	2	28	12
	17.700 R	96	93	85	74	56	45	39	33	27	23	3	35	20
	17.760 L	98	95	85	76	61	50	44	39	32	28	8	33	16
C2	17.840 C	99	92	86	72	59	51	43	34	28	23	19	29	13
Subgrade	17.900 L	nil	98	91	83	74	62	54	46	37	30	2	30	17
	17.960 C	99	97	91	84	74	61	53	45	35	29	nil	29	15
	18.030 R	95	92	80	68	59	53	47	38	27	23	5	23	9
	18.080 L	95	93	81	67	56	48	42	33	26	22	5	22	8
C3	18.580 C	96	93	86	79	70	58	51	45	35	29	10	29	16
Subgrade	18.600 L	96	94	88	81	71	59	52	46	36	28	8	28	12
	18.680 R	93	85	67	60	55	50	46	43	38	32	31	30	12
	18.800 C	87	79	64	54	49	46	43	40	35	30	38	45	23
	18.880 L	92	82	66	55	50	46	42	39	34	29	29	42	25
C4	19.360 C	83	71	51	40	35	32	30	28	23	20	nil	47	28
Subgrade	19.400 L	84	71	57	50	46	42	39	36	32	27	29	40	14
	19.440 R	83	68	51	42	39	36	34	32	28	23	35	42	23
	19.480 L	88	79	64	53	47	42	39	36	32	27	39	40	21
	19.592 R	92	89	79	69	59	53	49	44	39	34	9	45	23

Note: All gradings have 100% passing 37.5mm sieve
C = Centre, L = Left, R = Right

TABLE A6

Results of cube tests - construction

Section	Density (Kg/m^3)		28 day equivalent Cube Strength (MPa)	
	Average	Range	Average	Range
C1 North	2443	2415 - 2459	44.0	32.1 - 47.2
C1 South	2446	2411 - 2475	41.8	31.5 - 46.4
C2 North	2445	2426 - 2466	40.4	32.3 - 45.1
C2 South	2443	2407 - 2456	42.4	32.5 - 47.2
C3 North	2469	2428 - 2507	49.9	39.8 - 68.3
C3 South	2458	2421 - 2493	44.4	34.3 - 50.4
C4 North	2460	2404 - 2494	47.7	36.0 - 58.8
C4 South	2467	2431 - 2513	43.6	28.9 - 63.6
All Sections:	2454	2404 - 2513	44.2	28.9 - 68.3

TABLE A7

Results of beam tests - construction

Section	Density (Kg/m^3)		28 day equivalent Flexural Strength (MPa)		28 day equivalent Cube Strength (MPa)	
	Average	Range	Average	Range	Average	Range
C1 North	2443	2420 - 2467	4.82	4.17 - 5.45	48.7	27.8 - 58.2
C1 South	2460	2414 - 2518	4.46	3.36 - 5.07	43.6	31.4 - 51.9
C2 North	2442	2386 - 2492	4.51	3.76 - 5.06	38.5	32.8 - 43.9
C2 South	2445	2418 - 2496	4.61	3.97 - 5.56	47.4	32.1 - 59.2
C3 North	2469	2346 - 2528	4.79	4.17 - 5.74	52.8	35.5 - 66.8
C3 South	2472	2431 - 2516	4.64	3.95 - 5.48	49.1	36.9 - 82.4
C4 North	2483	2422 - 2526	4.85	3.77 - 5.37	51.8	34.7 - 62.8
C4 South	2480	2443 - 2528	4.71	3.92 - 5.55	49.8	28.4 - 68.3
All Sections:	2462	2414 - 2528	4.67	3.36 - 5.74	47.7	27.8 - 82.4

TABLE A8

Core test results

Section	No. of tests	Density (Kg/m^3) Average	Density (Kg/m^3) Range	90 day equivalent Cube Strength (MPa) Average	90 day equivalent Cube Strength (MPa) Range
C1	3	2448	2416 - 2512	50.5	40.4 - 59.1
C2	2*	2485	2482 - 2487	60.7	58.8 - 62.5
C3	4	2427	2410 - 2451	57.8	52.8 - 62.3
C4	3	2444	2423 - 2459	57.8	52.3 - 60.9
All sections:		2451	2410 - 2512	56.7	40.4 - 62.5

* These cores had a lower than recommended height to diameter ratio.

TABLE A9

Cracking summary

Section	No. slabs	Average concrete thickness		Longitudinal 87	88	89	90	91	92	Transverse 87	88	89	90	91	92	Corner 87	88	89	90	91	92	Failed slabs 1992 No.	%
C1 North			number:	0	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	2	2	0	
			length (m):	0	0	0	0	0	0	0.5	1	1	1	1	1	-	-	-	-	-	-		
	128	173 mm																					1
C1 South			number:	0	0	0	0	0	0	2	2	2	2	2	2	0	1	1	1	1	1	1	
			length (m):	0	0	0	0	0	0	7	7	7	7	7	4	-	-	-	-	-	-		
C2 North			number:	0	0	0	0	0	1	4	6	6	6	6	13	0	2	2	2	2	5	6	
			length (m):	0	0	0	0	0	1	9	15	15	15.5	15	30	-	-	-	-	-	-		
	128	124 mm																					7
C2 South			number:	1	1	0	0	0	0	1	1	1	1	1	4	3	3	2	3	3	4	3	
			length (m):	4	4	0	0	0	0	3.5	3.5	3.5	3.5	3.5	11	-	-	-	-	-	-		
C3 North			number:	0	0	1	1	1	1	7	7	7	8	8	5	6	6	6	7	9	7	1	
			length (m):	0	0	0.3	0.3	0.3	0.3	8	8	9	10	10	8	-	-	-	-	-	-		
	152	186 mm																					3
C3 South			number:	0	0	0	0	0	0	4	4	4	4	4	5	5	5	5	5	5	5	4	
			length (m):	0	0	0	0	0	0	13	13	14	14	13	13	-	-	-	-	-	-		
C4 North			number:	0	0	1	1	1	3	5	6	7	7	7	12	0	0	1	1	3	3	7	
			length (m):	0	0	0.3	0.3	0.3	2.5	13	15	17	18	18	36	-	-	-	-	-	-		
	163	151 mm																					5
C4 South			number:	0	0	0	1	1	2	2	2	2	2	2	5	2	2	2	3	3	4	1	
			length (m):	0	0	0	0.3	0.3	1.5	7	7	7	7	7	10	-	-	-	-	-	-		

TABLE A10

Joint defect summary - sealant condition

Section		Bad 88	Bad 89	Bad 90	Bad 91	Bad 92	Fair 88	Fair 89	Fair 90	Fair 91	Fair 92	Good 88	Good 89	Good 90	Good 91	Good 92
C1 (n=64)	No:	0	2	1	14	60	7	25	50	43	4	57	37	13	7	0
	% :	0%	3%	2%	22%	94%	11%	39%	78%	67%	6%	89%	58%	20%	11%	0%
C2 (n=64)	No:	1	1	4	4	35	4	29	46	50	29	59	34	14	10	0
	% :	2%	2%	6%	6%	55%	6%	45%	72%	78%	45%	92%	53%	22%	16%	0%
C3 (n=75)	No:	0	13	12	-	66	18	43	61	-	9	57	19	2	-	0
	% :	0%	17%	16%	-	88%	24%	57%	81%	-	12%	76%	25%	3%	-	0%
C4 (n=82)	No:	2	2	5	13	63	20	55	62	59	17	60	25	15	10	2
	% :	2%	2%	6%	16%	77%	24%	67%	76%	72%	21%	73%	31%	18%	12%	2%

TABLE A11

Joint defect summary - spalling & stepping

Section	Spalling at joints (No.) 1992	Spalling at cracks (No.) 1992	Stepping >5mm at joints & cracks (No.) 1992
C1 North	0	0	0
C1 South	1	0	0
C2 North	0	4	0
C2 South	1	2	0
C3 North	4	0	0
C3 South	0	2	0
C4 North	6	5	4
C4 South	0	1	1

TABLE A12

Roughness measurements

Section	Concrete Design	Profile Beam Reference Roughness (mm/km)* 1987	1988	1989	1990	1991	1992
C1 North	150mm on	2311	2398	2467	2764	3013	2302
C1 South	sub-base	2322	2597	2432	2486	3152	2408
C2 North	125mm on	2781	2840	2921	3071	2956	2837
C2 South	sub-base	2534	2568	2538	2874	2686	2519
C3 North	175mm	2342	2508	2367	2504	2313	2284
C3 South	Dowelled	1997	2242	2346	2076	1985	2006
C4 North	125mm	2637	2650	2576	2637	2394	2559
C4 South		2776	2833	2827	2827	2564	2705

* Using TRL eqn: $RR = 2045 \times RMSD + 68 \times (RMSD)^2$
where RMSD = Root Mean Square of Deviation for the roughness measurements.

TABLE A13

Skidding resistance measurements

Section	Skidding Resistance Value					
	1987	1988	1989	1990	1991	1992
C1 North	109	88	107	120	106	77
C1 South	90	90	104	122	89	95
C2 North	118	95	104	117	108	75
C2 South	116	76	106	116	84	97
C3 North	100	74	98	114	94	71
C3 South	88	106	94	116	92	78
C4 North	94	109	113	124	95	68
C4 South	92	107	108	118	91	86
Average:	102	93	104	118	95	81

TABLE A14

Sand patch/texture depth results

Section	Average texture depth (mm)					
	1987	1988	1989	1990	1991	1992
C1 North	1.0	1.1	0.8	0.9	0.9	1.0
C1 South	0.9	1.1	0.9	1.0	1.0	1.0
C2 North	1.0	1.0	0.7	1.0	1.1	1.0
C2 South	1.4	1.4	1.2	1.4	1.4	1.3
C3 North	1.2	1.7	1.1	1.4	1.4	1.3
C3 South	1.1	1.0	0.7	0.9	1.0	0.9
C4 North	1.5	1.6	1.3	1.4	1.6	1.7
C4 South	1.3	2.1	1.2	1.5	1.5	1.7
Average:	1.2	1.4	1.0	1.2	1.2	1.2

TABLE A15

Benkelman beam & Logger deflections

Average Transient Deflection (mm x 0.01)

Section	Nearside wheelpath								Offside wheelpath			
	1988	1989	1990		1991		1992		1989	1990	1991	1992
	B	B	B	L	B	L	B	L	B	B	B	B
C1 North	18	14	19	(23)	20	(13)	29/22	(15)	16	20	24	31/26
C1 South	35	23	24	(18)	30	(24)	40/25	(34)	16	23	27	41/23
C2 North	18	16	21	(20)	24	(15)	31/29	(17)	20	21	29	36/38
C2 South	23	20	23	(21)	43	(39)	26	(10)	11	21	42	22
C3 North	13	14	14	(14)	20	(21)	17	(8)	15	15	23	19
C3 South	23	16	25	(30)	40	(37)	29	(19)	12	15	28	23
C4 North	47	24	55	(92)	36	(30)	33	(19)	12	40	35	38
C4 South	27	25	24	(19)	65	(69)	50	(42)	16	21	59	50
Average:	25	19	26	(30)	35	(31)	30	(20)	15	22	33	32

Note: x/y indicates a repeated run.
B = Benkelman Beam measurements.
L = Deflection Logger measurements.

TABLE A16

Rod and level survey measurements for slab C1-97

Position of level measurements on slab:

SHOULDER		
H	I	B
G	A	C
F	E	D

July 1992 Date:	Time	Height (mm) A	B	C	D	E	F	G	H	I
6	0615	131.0	163.5	116.2	59.0	88.3	106.0	161.5	209.2	192.6
6	0710	147.0	179.2	131.8	74.5	104.2	121.6	177.4	225.2	208.2
6	0800	127.3	160.0	112.8	55.3	84.7	102.4	158.0	206.0	189.0
7	0835	78.3	111.5	64.0	7.0	35.8	54.0	109.2	157.6	140.2
6	1055	196.5	231.0	183.2	126.5	155.0	174.0	229.0	277.5	259.2
7	1140	48.5	82.5	34.0	-21.5	6.0	25.0	80.0	129.0	110.5
6	1315	134.4	168.8	120.3	64.0	92.0	111.3	166.2	214.8	196.7
7	1320	103.2	138.0	89.4	33.2	61.2	80.2	135.8	183.8	166.0
4	1450	125.8	160.0	112.5	55.3	83.5	103.0	157.3	206.0	188.0
6	1525	196.3	230.2	182.2	126.0	154.5	173.5	229.0	277.0	259.0
4	1545	96.2	130.0	82.2	25.2	54.0	72.2	127.4	176.0	157.5
6	1630	185.0	219.0	171.0	114.3	142.8	161.5	216.8	265.0	247.3
6	1720	173.3	207.0	159.0	102.3	131.0	149.3	205.0	253.2	235.5

TABLE A17

Concrete temperature measurements

TIME	Temperature of slab C4-163 on 6-7 July 1992 TOP (°C)	MID DEPTH (°C)	BASE (°C)
01:00	10.4	14.3	15.9
02:00	9.8	13.7	15.3
03:00	9.3	13.1	14.7
04:00	8.8	12.6	14.3
05:00	8.3	12.1	13.8
06:00	7.8	11.7	13.4
07:00	7.6	11.2	13.0
08:00	9.1	11.2	12.7
09:00	11.2	11.6	12.5
10:00	14.4	13.1	12.7
11:00	17.5	14.9	13.5
12:00	20.0	16.8	14.5
13:00	22.1	18.7	15.8
14:00	23.3	20.2	17.0
15:00	24.1	21.5	18.2
16:00	23.6	22.1	19.1
17:00	21.9	22.1	19.7
18:00	19.1	21.3	19.9
19:00	17.1	20.2	19.6
20:00	15.6	19.0	19.1
21:00	14.3	18.0	18.5
22:00	13.2	17.0	17.9
23:00	12.2	16.1	17.2
24:00	11.3	15.3	16.6
MIN:	7.6	11.2	12.5
MAX:	24.1	22.1	19.9

APPENDIX B: COST ANALYSIS OF CONCRETE PAVEMENTS IN ZIMBABWE

- **condensed from a paper by W C Kuwaza, Research Branch, Ministry of Transport, Zimbabwe.**

Construction costs

Five civil engineering contractors tendered for the project in March 1955. The winning tender was priced at Zim $263,134 but the final price proved to be Zim $230,076.

The construction of the concrete slabs cost Zim $18,000 more than originally estimated. Of this additional cost about Zim $13,000 came from the 11% increase in concrete volumes brought about by the problems with levels. The rest of the increase was caused mainly by increases in the price of cement and aggregate. The rise in the cost of cement increased the cost of the concrete pavement (including flexible shoulders) by about Zim $6,000 per km. The corresponding increase for flexible pavements (2% cement in base 1) is about Zim $1,000 per km. The net increase due to this factor is thus Zim $5,000 per kilometre comparing concrete and flexible pavements.

Comparison of construction costs - concrete vs flexible pavements

Table A18 gives a detailed cost breakdown of the concrete pavement construction. Estimates for structures, earthworks, base 2 construction and shoulder construction were obtained from the Road Design Branch of the Ministry for the 20 - 62 km section of the Mvuma - Gweru road being funded by the African Development Bank (ADB). The costs were calculated by simple proportion for the length involved in the concrete pavement project. The estimated cost of flexible pavement per kilometre was taken from the aforementioned ADB funded section.

The following comparisons can be made:-

cost/square metre of concrete slab	Zim $23
cost/square metre of completed concrete pavement	Zim $28
cost/square metre of flexible pavement	Zim $23

The total cost per square metre of a concrete pavement would be lower than the above figure if done on a large scale as the cost of preliminary items would be lower per unit length of completed pavement. Using the above figures as a basis, concrete pavements cost 22% more at construction than flexible pavements. In the United Kingdom the construction costs depend on local conditions with concrete pavements being cheaper in certain areas and more expensive in others. In the Philippines the cost difference between the two methods of construction is not significant. The same state of affairs was also noted in the Caribbean Island of Virgin Gorda (Parry, 1985).

Comparison of overall costs of concrete vs flexible pavements

In Western Europe and the USA, problems of ascertaining which pavement is the cheaper arise from the different interest groups or lobbies. The concrete lobby emphasizes the low periodic maintenance costs of this type of pavement while playing down the usually higher cost of construction. The flexible lobby on the other hand highlights the usually lower construction cost while playing down the usually more expensive maintenance aspects. The two sides often use different methods of economic analysis. The Portland Cement Association of USA have recommended the use of the "Cash flow method" which seems to favour concrete roads, in that the higher costs of construction are not discounted over the life of the road. This coupled with the low maintenance costs makes concrete roads cheaper.

The flexible pavement lobby tend to favour the "present worth" method by using an appropriate discount rate on the cost of construction which often results in their pavement being cheaper than concrete.

The Permanent International Association of Road Congresses (PIARC) have shown in recent analyses by the committee on concrete roads that no single country has clearly shown the advantages in terms of costs between flexible and concrete pavements.

In Zimbabwe, Makoni (unpublished) used a present worth type method, which showed that concrete roads were marginally cheaper for the most heavily trafficked pavements but more expensive for the lower traffic categories. The Author has used the cash flow method on the pavement at Mvuma - Gweru and found concrete pavements to be substantially cheaper than flexible pavements (Table A19). These conflicting results give a vivid illustration of the problems of economic comparison.

TABLE A18

Cost of rigid and flexible pavements at construction

A.	Total cost for 1.42km concrete pavement	Zimbabwe $
	Preliminary Items	39,650
	Structures (culverts)	40,910
	Earthworks	76,850
	Pavement Construction for 640m base 2 natural:	
	(a) Clear gravel pit	240
	(b) Remove overburden	260
	(c) Stockpile base 2	2,400
	(d) Load, haul up to 1km, control compact	3,460
	(e) Overhaul pavement material	3,520
	(f) Water for controlled compaction	890
	Prime (used on C4 only)	1,340
	Construct shoulders, prime seal with bitumen	23,000
	Construct asphaltic concrete at transition with flexible pavement (6m 3 total)	2,000
	Construct concrete slab	190,430
	Miscellaneous (Fencing, signs, markings)	12,340
		TOTAL 397,290
		TOTAL PER Km 280,000
B.	Cost of flexible pavement Estimated cost per km	230,000
C.	Compare cost of constructing rigid and flexible pavement Additional funds required for rigid pavement per km	Zim $ 50,000
	Rigid pavement costs 22% more than flexible pavement	

TABLE A19

Cash flow method of payment for concrete and flexible pavements:
Period of comparison 25 years at 10% annual increase in prices

A.	Concrete Pavement per kilometre	Zimbabwe $
	Initial cost	280,000
	Total routine maintenance cost:	106,560
	At 10 years periodic maintenance:	18,160
	At 20 years periodic maintenance:	47,090
		TOTAL 451,810
B.	Flexible Pavement per kilometre	
	Initial cost	230,000
	Total routine maintenance cost:	106,560
	At 10 years reseal:	32,420
	Residual life of reseal at 20 years	0
	Reconstruction at 20 years:	739,970
	Salvage value of reconstruction at 25 years:	
		554,980
		553,970

Therefore flexible pavement costs 23% more than concrete pavement.

NOTES ON ASSUMPTIONS MADE

1. Concrete pavement has a life of 25 years; flexible 20 years.
2. Routine maintenance costs are the same for both types of pavement (Zim $985 p.a. based upon National Transport Study 1985).
3. Periodic maintenance of concrete road cost Zim $7,000 p.a. at present prices, this being restricted to joint sealing. Seal every 10 years.
4. Reseal of flexible pavement costs Zim $12,500 at present prices and last 10 years.
5. Reconstruction of flexible pavement costs Zim $110,000 at present prices. This is a mean value from asphaltic concrete, granular overlay and reconstruction costs as given in National Transport Study 1985.

Printed in the United Kingdom for HMSO
Dd K 63500 8/93 C5 G542 10170