Comparison of Accelerated Pavement Test facilities in New Zealand and Australia

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Contents

Executive Summary

As part of an Austroads-funded project under the 1997/98 National Strategic Research Programme (NSRP), reviews were conducted of the operation of the Australian 'Accelerated Loading Facility' (ALF), owned and operated by ARRB Transport Research, and the New Zealand 'Canterbury Accelerated Pavement Testing Indoor Facility' (CAPTIF), owned and operated by Transit New Zealand. The first step in this process was the preparation of reports describing both facilities, their operation and management and a summary of trials conducted.

During 1998/99, Transfund commenced a cross experiment between CAPTIF and ALF based on the specific characteristics of each facility. As part of the programme, a quantity of crushed rock, previously tested by ALF, was shipped from Australia and tested at CAPTIF. The aims of the Transfund project were to:

- establish a procedure for the systematic exchange of information and expertise between ALF and CAPTIF,
- identify points common to the two research programmes,
- develop a method to ensure harmonisation of measurement methods, and
- conduct parallel experiments based on the specific characteristics of each facility.

This project addressed these issues and carried out the cross-test, using CAPTIF, on a granular material previously tested by ALF. The project also described both facilities and summarised and evaluated research conducted up to 2000.

Background
CAPTIF was commissioned in 1986 and since then has applied 8,400,000 load cycles over 13 separate trials. ALF was commissioned in 1984 and has applied over 25,000,000 load cycles to 200 pavement sections in 25 trials. Both facilities have addressed the following research areas in various tests; traffic loading, asphalt and binders, unbound, recycled and stabilised road making materials, pavement design and structural behaviour. In addition ALF has also conducted research in road surfacings and markings, construction, maintenance and rehabilitation methods and asset management.

Research findings
The performance of the granular material when it was tested using ALF and CAPTIF was similar, which suggests that the results of testing at either facility can be confidently translated, provided that other factors can be accounted for. These include, in particular, the environment, i.e. height of water-table, temperature and subgrade conditions. This implies that material need not be shipped between New Zealand and Australia provided the appropriate materials can be located in the testing country.

The controlled environment at CAPTIF makes it suitable for tests that require high numbers of load applications, because the deterioration of the pavement can be directly related to the number of load applications (all other points being equal).

Because the CAPTIF pavement is constructed under cover and contained in a tank, there is no risk of uncontrolled water entering the pavement structure.

In general, accelerated pavement testing (APT) provides a useful link between laboratory testing on small samples and in-service pavements. APT is better than small scale testing as the loading and pavement structure are more realistic and APT is better than in-service testing as the load and climate are either controlled or measured. However APT cannot easily or quickly replicate seasonal changes in the moisture content of the pavement, nor can it accelerate the ageing of the bituminous materials. The scope of APT is usually limited by time and budget constraints, it is only possible to test a very limited number of combinations of pavement materials, subgrade conditions, layer thicknesses, climatic conditions and loading conditions.

Recommendations

The authors propose the uniform exchange of information to help ensure that any research conducted is timely and not duplicating effort and also to assist in the practical implementation of any findings.

Cooperation using the ALF and CAPTIF facilities for investigating pavement behaviour is desirable because of the diversity of the factors to be taken into account in pavement engineering, and the need to understand the mechanism of pavement deterioration. Coordination is also desirable if an attempt is to be made to reduce the number of different approaches to this problem and to optimise the use of limited resources.

New Collaborative Work

A programme of work currently being funded by Transfund and Austroads should lead to a series of parallel trials, using both CAPTIF and ALF, from 2001/2002. The main objective of this project is the development of a more rational basis for assessing the potential impacts of future vehicle configurations on the road pavement asset over a range of typical operating conditions, especially with respect to unbound granular pavements with thin bituminous surfacings, which comprise 95% of Australasia's sealed road network.

Abstract

This report presents the findings from a review of the operation and completed projects conducted at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) and the Australian Accelerated Loading Facility (ALF). A test was undertaken at CAPTIF in 1999 where a granular material was imported from Australia and tested under CAPTIF loading. The material had been previously tested by ALF. The results from this test show that the performance of the material was similar under loading by both devices, allowing for differences in the testing environments. The strengths and weaknesses of both facilities are compared and the possibilities for collaboration and technology transfer between the two facilities are explored.

1. INTRODUCTION

As part of an Austroads-funded project under the 1997/98 National Strategic Research Programme (NSRP), reviews were conducted of the operation of, and research conducted using, the New Zealand 'Canterbury Accelerated Pavement Testing Indoor Facility' (CAPTIF), which is owned and operated by Transit New Zealand, and the Australian 'Accelerated Loading Facility' (ALF), which is owned and operated by ARRB Transport Research.

The aims of the Transfund project were as follows:
- Establish a procedure for the systematic exchange of information and expertise between ALF and CAPTIF.
- Identify points common to the two research programmes.
- Develop a method to ensure harmonisation of measurement methods.
- Conduct parallel experiments based on the specific characteristics of each facility.

This report addressed these issues, including the results of a cross-test conducted, using CAPTIF, on a granular material previously tested by ALF.

The first step in this process was the preparation of reports describing both facilities, their operation and management and a summary of trials conducted to that date (Pidwerbesky and Steven 1998; Sharp, Johnson-Clarke and Vuong 1998). Sections 2 and 3 of this report describe in detail the New Zealand and Australian facilities respectively. A brief but comprehensive review of the trials conducted at both the New Zealand and Australian facilities are covered in sections 4 and 5 respectively.

A review of the use of accelerated testing results is presented in section 6. Section 7 provides a point by point comparison between the two facilities in terms of the measurements taken during projects, the instrumentation used, the topics addressed by completed trials and the capabilities of the two facilities.

During 1998/99, Transfund commenced a project involving an initial cross experiment between CAPTIF and ALF based on the specific characteristics of each facility and testing, as part of the programme, a crushed rock previously tested by ALF and shipped from Australia. The response of the pavement to loading, and pavement performance data collected, was to be compared as a means of assessing the degree of compatibility between the two devices. Confirmation that the outputs from each facility are complementary will lead to an increased degree of confidence in the interpretation and extrapolation of results from other tests. This would result in substantially improved cost efficiencies because the results from a greater range of tests could be correlated, and depending on the specific issue being addressed, reduced testing would result. The results of the trial undertaken at CAPTIF and the original ALF trial are reported in section 8.

The conclusions that were drawn from the review of the two facilities, the test programmes and the cross trial are presented in section 9, along with an outline of how the benefits from both programmes could be maximised without duplicating work in either country. In section 10 several recommendations are made that would improve the construction of test pavements at the CAPTIF facility.

Finally, brief details of a programme of work currently being funded by Transfund and Austroads are presented which should lead to a series of parallel trials, using both CAPTIF and ALF, from 2001/2002. The main objective of this project is the development of a more rational basis for assessing the potential impacts of future vehicle configurations on the road pavement asset over a range of typical operating conditions, especially with respect to unbound granular pavements with thin bituminous surfacings, which comprise 95% of Australasia's sealed road network.

2. Development of the CAPTIF Facility

The first New Zealand accelerated loading facility was constructed in 1969 (Williman and Paterson 1971). The first machine was used for a number of pavement research projects, and, in 1983, finally became unserviceable. An assessment of the need for a new, improved accelerated pavement loading facility identified four primary research priorities:

- the performance of aggregates, such as marginal materials,
- modified designs for surfacings, especially chip seals,
- evaluating pavement design assumptions by collecting data describing the long-term performance of pavements, and
- the relationship between vehicle loading conditions and the deterioration of pavements for a wide spectrum of pavement and loading characteristics.

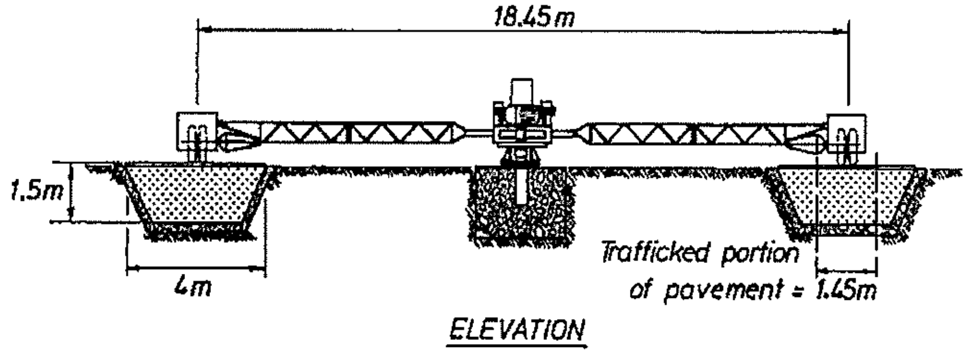

Figure 1.1 General schematic of CAPTIF

A circular test track in which full-scale pavements could be constructed, and a loading apparatus capable of imposing realistic dynamic heavy vehicle loading, was selected for the following reasons:

- The machine can be operated continuously without being interrupted for direction changes, thereby greatly increasing the rate of loading.
- After initial acceleration, the speed of the loading system can be kept constant for long periods of time, or varied, depending on the requirements of specific projects.
- Circular tracks can be divided into a number of either annular rings or longitudinal segments, each containing a pavement with some unique characteristics, and all segments can be tested simultaneously under the same or varying loading conditions.
- The configuration of each loading assembly in a multi-armed machine, which means that tyre type and pressure, axle numbers and weight, and suspensions and loads, can be altered so that the response of the same pavement under various loading conditions can be determined.

- The interaction of the pavement and the vehicle dynamics can be examined using a combination of unsprung and sprung masses possessing realistic damping characteristics.

CAPTIF is housed in a hexagon-shaped building that is 26 m wide and 6 m high. An annular concrete tank, 1.5 m deep and 4 m wide, confines the bottom and sides of the track (figure 1), enhancing the control of moisture content in the sub-surface systems and drainage. The track has a median diameter and circumference of 18.5 m and 58.1 m respectively. Normal field construction and compaction equipment is used in the facility. The main feature of CAPTIF is the Simulated Loading and Vehicle Emulator (SLAVE), which is described in more detail below.

2.1 Simulated Loading and Vehicle Emulator (SLAVE)

SLAVE was designed for the accelerated testing and evaluation of subgrades, pavements and surfacings by replicating the effect on the pavement of actual road traffic conditions. An elevation view of SLAVE is presented in (figure 2) and its characteristics are listed in table 1. A sliding frame within the central platform is moved horizontally a maximum of 1 m (from stop to stop) by two hydraulic rams; this radial movement produces multiple wheelpaths. The base elevation can be altered by up to 150 mm, to maintain the dynamic balance of the machine if the pavement surface level changes due to either vertical deformation in the pavement or an overlay being applied.

Each vehicle consists of the axle, which is driven by a hydraulic motor, a suspension, a frame, instrumentation, and standard wheel hubs and truck tyres (figure 2). The SLAVE vehicles can carry either single or dual tyres and their loads can be adjusted to between 21 and 60 kN (42-120 kN axle loads) by adding or removing steel weights. The suspensions can be either multi-leaf steel spring, parabolic steel leaf spring or air bag, and each vehicle can carry the same or a different suspension for simultaneous testing. The speed can be varied between 0 and 50 km/h, and varied during operation. The vehicles can be moved slowly, and positioned at any location on the track, using a hand-held, infra-red remote control.

Table 1.1 Characteristics of the Simulated Loading and Vehicle Emulator (SLAVE)

Item	Characteristic
Test Wheels	Dual or single tyres; standard or wide-base; bias or radial ply; tube or tubeless; maximum overall tyre diameter of 1.06 m
Mass of Each Vehicle	21 kN to 60 kN, in 2.75 kN increments
Maximum Loading	5 ESAs/vehicle = 10 ESAs/complete revolution
Suspension	Air bag; multi-leaf steel spring; single or double parabolic
Power drive to wheel	Controlled variable hydraulic power to axle; bi-directional
Transverse movement of wheels	1.0 m centre-to-centre; programmable for any distribution of wheelpaths
Speed	0-50 km/h, programmable, accurate to 1 km/h
Geometry of Track	Radius of Travel 9.2 m; overall length 58 m; width 4 m
Loading	Maximum number of passes applied during one hour: 1700 Theoretical minimum duration to apply 10^6 ESAs: 5 days

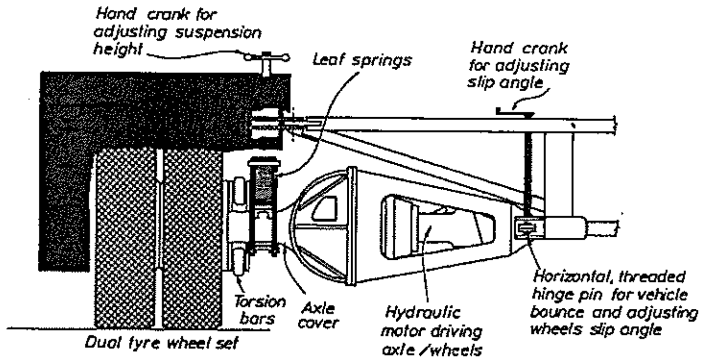

Figure 1.2 Detail of the SLAVE

SLAVE operations are directly controlled by an on-board PLC system. The PLC system is configured by a supervisory program running on an external or on-shore computer. Whenever a parameter is to be altered, the new command is sent by the external computer through a Spread Spectrum RF modem link.

Testing routines can be programmed in terms of start/stop times, distance or revolutions to be run, travelling speeds, and tracking pattern of wheelpath positions, to name only a few. Any combination of these may be included in a programmed testing routine because the SLAVE software will use default values for those items not defined in the shore computer program. Manual control can be imposed when desired, to override the current program. In addition to conventional hydraulic pressure, electrical current and motor over-load devices, the SLAVE electronics continually scans the safety monitors, and if a condition occurs which requires human inspection, brakes the vehicles to a stop.

2.2 Instrumentation and Data Acquisition

Since the SLAVE was commissioned in 1987, electronic systems have been introduced that measure dynamic and residual strains and displacements, surface profiles, rebounds and temperatures in the pavement and subgrade. The CAPTIF Deflectometer, which is a modified version of the Geobeam device developed by Tonkin and Taylor Ltd of Auckland and resembles a Benkelman Beam, measures the surface deflection of a pavement under a wheel load. The Deflectometer probe is positioned between the tyres of a dual-tyred wheel and, as the wheel is moved away, the rebound of the pavement is measured, to the nearest 0.01 mm, every 50 mm of horizontal movement. There are no moving parts on the device: an electro-magnetic gap-measuring sensor at the end of the beam measures the vertical distance between the sensor and a steel disc placed on the pavement surface. A separate, associated device measures the horizontal movement of the wheel. Surface deflections can also measured by a Falling Weight Deflectometer (FWD). An external consultant owns the FWD.

The CAPTIF Profilometer measures transverse surface profiles using similar electronics. The Profilometer consists of a braced aluminium beam, 4.4 m long, supported at each end by adjustable feet. An aluminium carriage is driven along the beam by an electric motor and drive chain. The carriage holds a Linear Variable Displacement Transducer (LVDT) with a jockey wheel riding along the pavement surface. Vertical displacement is recorded every 25 mm of horizontal travel of the carriage.

The output signals are digitised by electronics contained within the devices, and the digital data are captured by a Psion hand-held computer. A DIPStick profiler is used to measure the longitudinal surface profiles, for roughness surveys. Longitudinal surface profiles are also measured by a laser and accelerometer device mounted on one of the SLAVE units. The laser profiler was developed by ARRB TR Ltd. The output from temperature probes installed in the pavements are automatically recorded hourly by a Taupo F-10-24K-48A data logger. For the purpose of measuring the dynamic loads being applied by the axles, a Hewlett Packard 3852S microprocessor-based unit and computer capture data signals from accelerometers and displacement transducers mounted on the chassis and axles of each vehicle. The Hewlett Packard 3852S unit is controlled by an industrial PC which is mounted on the SLAVE, which in turn is controlled by remote-control software operating over a RF LAN link. This system enables data acquisition to be carried out while the vehicles are running at speeds up to 50 km/h without having to stop the machine.

The soil strain measuring system determines minute strains (100 µm/m) with good resolution (±50 µm/m) using Bison Coil type strain sensors. The sensors use the principle of inductance coupling between two free-floating, flat, circular wire-wound induction coils coated in epoxy, with a diameter of 50 mm. One of the two discs acts as the transmitter coil, creating an electro-magnetic field which induces a current in the receiving coil. The magnitude of the induced current is inversely proportional to the spacing between the two coils. The gauge length is the separation distance between each paired coil. The discs are installed during the formation of the subgrade and the overlying layers, to minimise the disturbance to the materials.

The original CAPTIF strain-measuring system was a modified prototype of the Saskatchewan Soil Strain/Displacement-measuring system (SSSD) developed by Saskatchewan (Canada) Highways and Transportation. The CAPTIF system used a dedicated computer containing a specially-built General-Purpose Input/Output (GPIO) board, circuit boards, rectifiers, amplifiers and assembler code written specifically for this application. Each sensor in an array was scanned simultaneously when triggered, for every 30 mm of vehicle travel, so that a continuous bowl shape of strain versus distance travelled was obtained. In 2000 a new soil strain Emu system was purchased from Nottingham University to replace the SSSD system. The SSSD system was replaced because of reliability problems with the hardware and commercially manufactured coils could no longer be obtained. The new system still uses the same inductive coil principal as before, but the excitation, reading and decoding hardware is different. An unlimited number of coil pairs can now be read as each coil is multiplexed through the Emu unit. The supply of coils is also guaranteed as they are now manufactured by CAPTIF staff. The multiplexing and computer interface equipment are standard commercial parts and the controlling software is written using the LabView computer program.

For the current project (C13 in table 4.1), a number of Dynatest Soil Pressure Cells were purchased to measure the dynamic soil stress in the subgrade. Sufficient gauges were purchased to enable the stress to be measured in three dimensions at three separate locations in the pavement. The gauges are read through the same multiplexing system and software that is used for the Emu soil strain equipment.

3. Development of ALF

By the late 1970s it was realised that, as the emphasis on the maintenance and rehabilitation of existing facilities continued to increase, there was a need to determine cost-effective asset management strategies, improve rehabilitation design procedures and validate the use of innovative materials. The commonly employed empirical design procedures, typically derived from experimental pavement studies such as the AASHO Road Test, had proven to be inadequate for these increased vehicle numbers, loads and tyre pressures. Whilst long-term pavement performance studies, such as the current SHRP LTPP study, are an integral part of any pavement research strategy, it was recognised that such studies needed to be supplemented with accelerated full-scale testing and laboratory materials characterisation, allied to the use of mechanistic pavement design procedures, if the results were to become available in a reasonable timeframe and if the optimum use were to be made of limited financial and technical resources.

Early studies of pavement systems, and associated field studies, addressed the issues of load and layer equivalency in an attempt, principally, to define the levels at which unbound pavements were no longer adequate. This early research demonstrated the complexity of material response and suggested that some form of rolling wheel track was required to provide the link between long-term field trials and the characterisation of the pavement materials in the laboratory. This resulted in the construction of a quarter-scale linear test track which operated for some years and produced some useful results but also exposed the limitations of conducting this type of research with small-scale models (Sparks and Davis 1978). The results of the NAASRA Economics of Road Vehicle Limits (ERVL) study (Fry *et al.* 1976), especially with respect to axle load distribution and pavement damage predictions, associated studies of the dynamic performance of suspensions (Sweatman 1980) and Australian State Road Authority (SRA) experience in major highway construction at that time also provided input into future research needs.

Whilst the ERVL review of the costs and benefits of heavy vehicles led to predictions of the impacts, costs and benefits of increased axle loads and different configurations, the predictions were based on limited local and overseas experience with heavier vehicles, higher traffic volumes, thicker pavements and stronger materials than those typical of Australian conditions. During the late 1960s and 1970s, many kilometres of high standard highway had been constructed, with the pavement designs and construction standards adopted being based on previous experience in the use of unbound bases with thin bituminous surfacings. Under some conditions the performance of these new pavements was disappointing.

A NAASRA Working Group review confirmed the need for research into pavement materials and their structural response to traffic loads, and led to the conclusion that these issues must be studied under full-scale conditions with at least the maximum legal load being applied by a moving wheel to a pavement composed of typical materials and constructed to normal dimensions using normal construction practice. It was noted that 'full-scale testing methods' could embrace a number of aspects, including the investigation of existing pavements under normal highway traffic, the use of controlled traffic on trial sections, dynamic load simulation and the use of accelerated testing facilities. The Working Group considered these options and concluded the following:

- The use of test sections in the road system under normal traffic was seen to be inadequate on its own because of the long time (10 to 20 years) required to obtain meaningful results and the uncontrolled nature of both the traffic and the environment. The provision for the examination of the effects of higher loads could only be made in 'off-road' situations where the rate of loading would be slow. (At that time, of course, procedures for measuring vehicle and axle loads without delaying the traffic were still being developed.)
- It was considered impractical to use 'under-designed' sections because, whilst the use of 'weak' test sections could provide distress data in a short period of time, early failures on the highway network would lead to unfavourable public reaction and, on heavily-trafficked routes, severe disruption to traffic and high costs occasioned by maintenance and reconstruction activities.
- Whilst loading could be controlled, and a shorter test duration achieved, by using vehicles of known load on a protected trial road section, the costs and time involved were large and the loads that could be applied were limited if standard highway traffic was used. Whilst this option was suitable for smaller-scale specific studies (e.g. Sharp and Armstrong 1985), it was not a viable option for large-scale, long-term studies addressing a wide range of issues of National interest.
- Whilst the historical analysis of road data could provide useful information on the 'present' road condition of pavements after various periods in use, difficulties are encountered in defining the 'initial' condition of the road, the effects of climate, the extent and effectiveness of maintenance, and the nature of the traffic over the life of the pavement. Other difficulties are associated with subgrade variability, the extent and adequacy of drainage and variability in material and construction quality.
- The use of laboratory materials testing allied to stress-strain analysis was also seen as inadequate, if no other testing was conducted, because it was not possibly to adequately define, or reproduce in the laboratory, the load and material conditions existing in the pavement. In addition, at that time, 'standardised' test methods, conducted on 'standardised' testing equipment' had not been developed and there was therefore no guarantee that results could be readily transferred to other situations.

The Working Group therefore concluded that the ideal test procedure should:

- test full-scale pavements constructed by normal plant – to provide for material and construction factors;
- use full-scale loads on moving wheels at normal speeds and frequencies – to provide for realistic stress patterns;
- allow for testing at accelerated rates and under higher loads – to provide results within a reasonable period of time and allow for future potential increases in vehicle mass; and
- be complemented by closely linked laboratory studies and field observations of trial sections in the road system.

3.1 Development of Final Specification for ALF

In drawing up the outline specification, the Working Group laid down its requirements without reference to the form of the facility. However, it was realised that no machine was ideal for all tasks and that some compromise would be required.

Opinion at one stage was strongly divided between a linear or a circular track, with some favouring a circular track because of its ability to conduct theoretically-based, multi-parameter evaluations of different pavements in a short period of time, and others favouring the approach used in South Africa with the Heavy Vehicle Simulator for its ability to 'proof test' actual roads. However, the cost of either solution, the difficulty of constructing a pavement in a circular track, the difficulty of housing such a track at ARRB headquarters, and the potential for the use of a linear configuration on real roads led to a compromise solution being adopted for further development. The ability to operate in the field was seen as a major advantage as it would potentially allow proof testing of different construction practices and in different climatic zones.

Another important issue was that funding for the facility was to be provided by all the State road agencies who, for that reason, would have a 'stake' in the facility and would wish to use it to address specific State-based issues as well as National issues. For that reason, it was imperative that the facility be relocatable, and the costs of providing permanent facilities for circular test tracks in every State were considerable compared with the costs of transporting the facility to every State. (The 'relocatable' nature of ALF has indeed proven to be one of the major reasons for its success over the years.)

An outline specification was developed but it was realised that some desirable features would have to be sacrificed in order to reduce cost and complexity. These included a reduction in maximum operating speed from the desired 80 km/h, because of the high forces generated when the wheel returned to its starting point, and the use of a simple wheel configuration without the provision for suspension and axle group variations, because the dynamic effects following the change of direction would result in a suspension not behaving 'normally' over a short test length.

Preliminary figures for the costs and resources of operating a full-scale test facility to the full performance specification, and conducting associated laboratory and field testing, were estimated at A$1 million for the manufacture of ALF and A$0.5 million for each year for operation. This was substantial, given that the total annual expenditure on pavements research in Australia at that time (early 1980s) was only A$1 million. However, it was a very small percentage of the national annual expenditure on road construction and maintenance at that time (A$2.4 billion). In addition, the existing research programme at ARRB at that time had a total budget of only A$5 million and it was conceded that the funding of the ALF programme would require both an increase in funding and a re-alignment of the research effort.

In terms of benefits/costs, it was estimated (Metcalf 1985) that the cost of a five-year programme, including initial capital costs, would be about A$6 million, which represented less than 0.1% of the annual road construction and maintenance budget. In other words, only about a 0.1% improvement in practice was required for ALF to be cost-effective.

3.2 Description of ALF

A schematic diagram of ALF is shown in figure 3.1, whilst the final specification is given in table 3.1. Rolling wheel loads, which can be varied in 10 kN increments from 40 kN to 90 kN, are applied, in one direction, to pavement test strips 12 m long at a constant speed of 20 km/h. The load assembly, which tracks linearly, is guided by the mainframe. The dual-wheel or single-wheel half single axle load (the single wheel assembly was commissioned in 1994/95) can be channelised or applied over a normal transverse distribution 1.4 m or 1 m wide (a new system is being developed which will allow any transverse distribution of load). The wheel is lifted off the pavement at the end of the cycle and supported by the mainframe on its return. The cycle time for each load is about 9 seconds, which corresponds to approximately 380 load cycles per hour or, depending on the percentage of operating time, about 50,000 cycles per week.

An automatic electronic control system operates the transverse distribution of load, logs operational data and monitors various machine states. Another system records the load and speed profiles during the loading cycle.

Initially the wheel was accelerated down a curved ramp and driven along the test length by electric motors on the wheels. At the end of the test length the trolley was decelerated as it ran up a second curved ramp and the wheels lifted clear of the pavement. In 1987 this system was replaced by a hydraulic system, as a result of which the dynamic peak forces were reduced by up to 50%, the length of uniformly

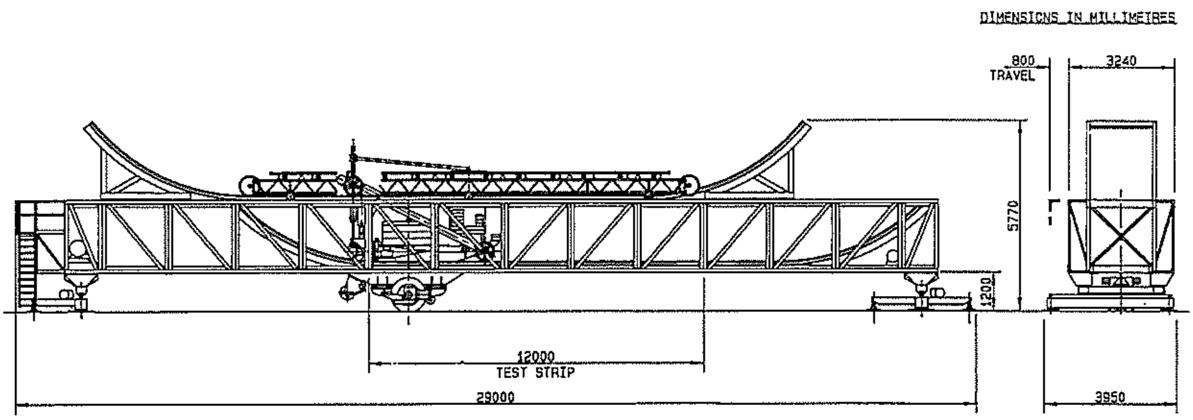

Figure 3.1 Schematic diagram of ALF

loaded test section was increased to the entire test length, rather than about two-thirds of its length, and noise and vibration, and hence maintenance costs, were substantially reduced (Kadar 1987).

Although ALF was constructed as a 'prototype' with an initial anticipated life of about five years and/or about 5 million cycles, only two major overhauls have been required since 1983 and the proportion of time that ALF is unavailable due to mechanical problems is now less than 10%. Routine maintenance of ALF forms a major part of any trial.

Table 3.1 ALF Specification

Test wheels	dual tyre: 12-22.5 Michelin "X" type, ZA pattern, 16 ply rating, tubeless; single: 15R22.5 and 18R22.5 tubeless, steel-belted radial
Mass of test wheel assembly	40 kN to 90 kN in 10 kN steps
Suspension for variable mass	air bag and shock absorbers
Power drive to wheel	2 x 11 kW electric geared motors, uni-directional operation, wheels off pavement on return
Transverse movement of test wheels	user programmable – typically a Normal distribution about 1.0 m or 1.4 m wide between outer edges of (dual) tyres
Test speed	nominally 20 km/h
Cycle time	approximately 9.5 seconds
Pavement test length	nominally 12 metres
Site constraints	max. grade: 1%; max. crossfall: 3%
Operation	automatic control system and fail-safe operation
Portability	readily detachable and transportable
Overall length of ALF	26.3 metres
Overall width of ALF	4.0 metres (operating); 3.2 metres (transport)
Overall height of ALF	5.7 metres (operating); 4.4 metres (transport)
Total mass of ALF	approximately 45 tonne

During transport between sites the mainframe is towed by a prime mover and mounted on a steerable bogie at the rear. Initially movement between test pavements at a particular site was achieved using cranes and/or air bags. More recently, a system of steerable bogies was installed onto the machine which allows it to be moved without the need for external assistance.

In September 1984 the US manufacturing rights were purchased by the Federal Highway Administration (FHWA) and two ALFs are located at the Turner-Fairbanks Research Centre at McLean, Virginia. Another ALF was later commissioned by the Louisiana Transport Research Centre. There are several differences between these ALFs and the original device, particularly in relation to the lifting mechanism: a system of camshafts is used compared to the hydraulic system adopted in Australia. Another ALF was manufactured in Australia for the Research Institute of Highways of the Peoples' Republic of China and has been operating in that country since 1987. This device is very similar to the original Australian ALF.

During 1994, a pavement heating system, based on the system developed by the FHWA, was manufactured and commissioned during an evaluation of the deformation properties of asphalt mixes. This system allows testing to be conducted at mid-depth pavement temperatures up to 60°C, at an accuracy of ±2°C.

3.3 Pavement Response and Performance Monitoring

Pavement performance and response to load is monitored during all the trials, the type of response recorded obviously dependent on pavement type and the aims of the trial. In general, the following testing is conducted:

- surface deflection measurements using the Benkelman Beam (BB) (figure 3.2) and the Falling Weight Deflectometer (FWD) (figure 3.3);
- deflections at the various layer interfaces using partial deflection gauges;
- vertical pavement (surface) deformation, and hence rut depth, using a Transverse Profilometer (figure 3.4), especially developed at ARRB TR to collect data associated with the ALF;
- within-pavement stresses and strains using pressure cells and H-bar strain gauges;
- daily maximum and minimum air temperature and rainfall and visual observations of pavement deterioration;
- surface cracking (figure 3.5); and
- moisture movement using Time Domain Reflectometry (TDR) gauges (figure 3.6).

This testing is complemented by extensive insitu testing (density, moisture content, etc.) and levels, etc., including trenching of the sites after trafficking is complete.

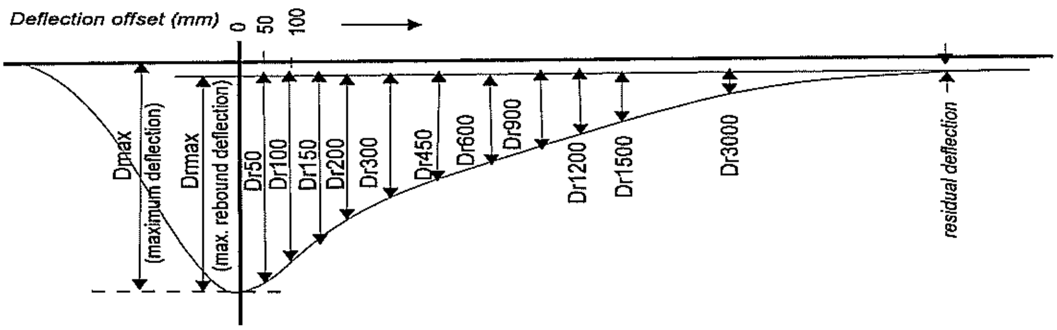

Figure 3.2 Typical Benkelman beam output and parameters monitored

Figure 3.3 View of Falling Weight Deflectometer

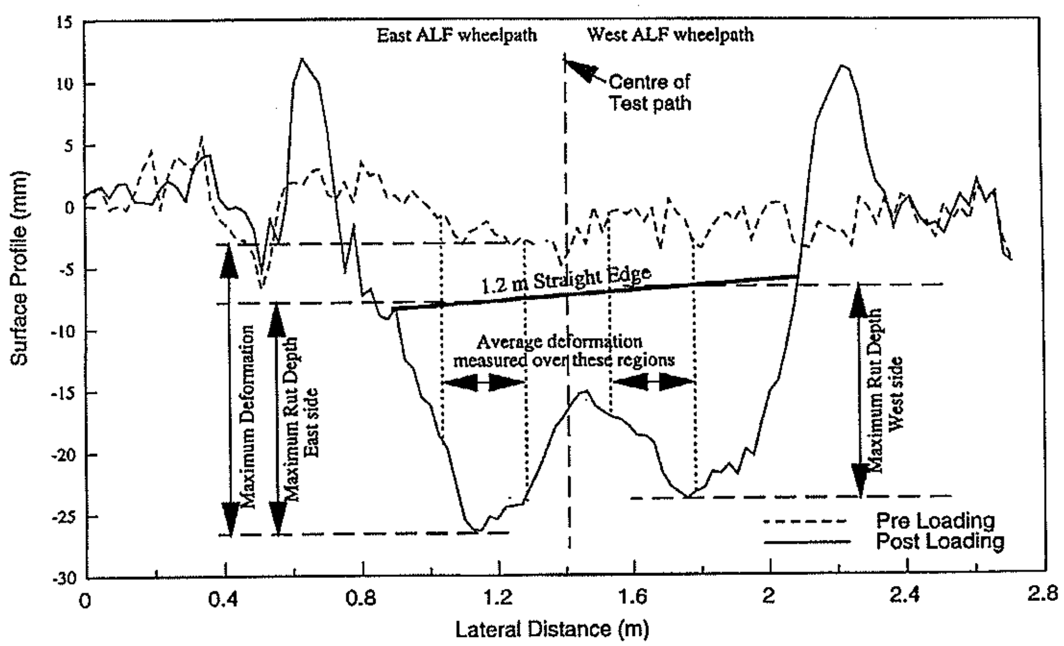

Figure 3.4 Typical transverse profilometer data

3.3.1 Data Acquisition and Processing System

The response of the test sections to the passing ALF wheel load (deflections) and permanent deformation is measured at regular intervals by displacement transducers connected to ARRB TR's Digital and Analogue Data Acquisition System (DADAS). The outputs of these devices, in the range of -1 to +1 V, are converted to digital units ranging from -2048 to +2047 and stored in solid-state memory of the DADAS from which they may be retrieved and stored on the hard disk of a Personal Computer. This allows both the quality of data to be controlled, and problems with the instrumentation and test equipment to be detected.

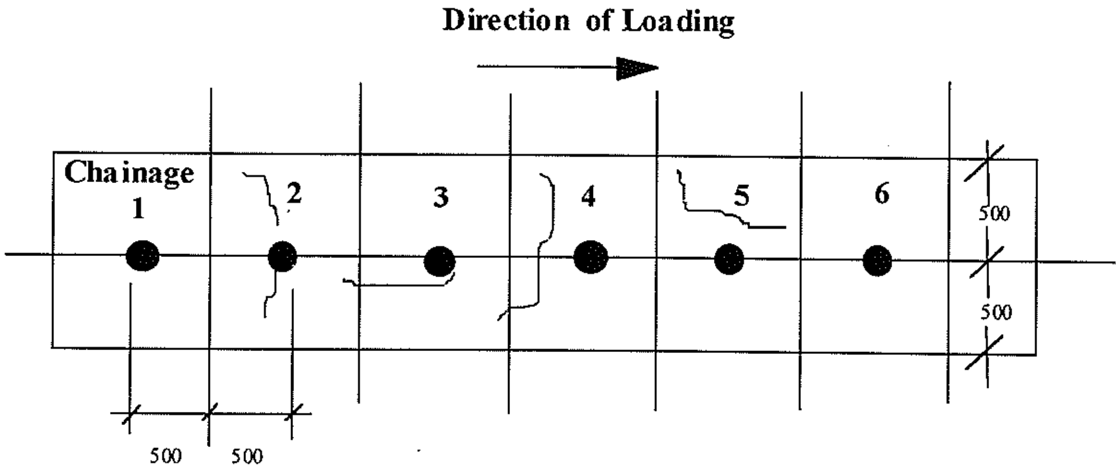

Figure 3.5 Explanation of crack severity

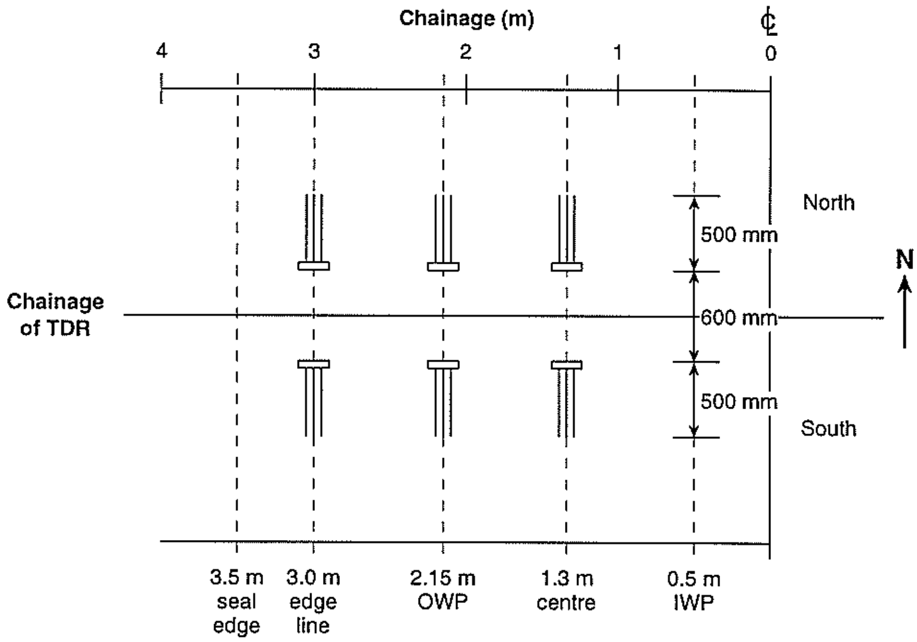

Figure 3.6 Typical layout of TDR gauges in a test section

After each set of measurements, the raw data are processed by applying the appropriate calibration factors to each of the instruments monitored.

The Benkelman Beam and Transverse Profilometer data are collected using displacement transducers measuring approximately every 25 mm. The BB readings are measured with the loading wheels travelling slowly across the trafficked pavement. Each data record can comprise several hundred data points and these are reduced using a variety of specially-developed programs. In order to reduce the data to a manageable size, only certain values are extracted and reported, for example:

- maximum rebound surface deflections (DR_0), plus rebound surface deflections at offsets from the maximum rebound surface deflection ($DR_{200}...DR_{1500}$) measured by the Benkelman Beam (see figure 3.2); and

- maximum permanent deformation along the trafficked strip, and average permanent deformation and maximum rut depth along each side of the trafficked strip measured with the ARRB transverse Profilometer (see figure 3.4).

The transverse profile data is measured with the ARRB transverse profile device every half metre along the trafficked strip, from ALF chainages 1 to 12, and processed through a computer program that first extracts the maximum deformation, and then makes an average of the ten readings at the centre of each of the ALF wheelpaths (see figure 3.4).

Surface crack development is recorded on clear plastic sheets, and crack length per square metre of pavement calculated, the square metre of pavement being defined as the metre-long interval centred on each chainage point (1 m apart) and extending half a metre each side of each chainage point (figure 3.5). The reported cracking severity therefore is related to the trafficked area rather than the entire lane area.

Time Domain Reflectometry (TDR) gauges, a system developed by CSIRO and modified by Queensland Department of Main Roads (QDMR) for use in pavements, are monitored to ascertain moisture movement within the pavement. A typical layout of the gauges is shown in figure 3.6. In this method, the volumetric moisture content is measured by the gauges in a non-destructive manner and the data is available immediately, provided the device has been calibrated for the site. The volumetric moisture content (q_v) is related to the gravimetric water content, w, by the expression:

$$q_v = w \,.\, (r_d/r_w)$$

where r_d = dry density of soil (t/m^3), and r_w = wet density of soil (t/m^3).

The degree of saturation, S_r, is related to the gravimetric water content and the volumetric moisture content by the following expressions:

$$S_r = w / \{(r_w/r_d) - (r_w/APD)\}$$

$$S_r = q_v / \{1 - (r_d/ APD)\}$$

where APD = apparent particle density (t/m^3).

3.4 Reporting and Analysis of Data

3.4.1 Data Reports

For all trials, reports are issued which present all the data collected during ALF trafficking. Deflection, deformation and cracking data are reduced to summary tables which present key data (date of testing, test site number and the data collected at each ALF chainage). FWD deflection bowl data, for example, will include the various deflection bowl parameters normalised to a standard contact stress, usually 700 kPa – though this, of course, varies depending on test conditions. Pavement and surface temperature are also recorded and presented. Accompanying the tables of data will also be figures comparing, for example, variation in maximum deflection with ALF chainage and change in deflection at a particular location over time and with the number of ALF loading cycles.

The philosophy behind the data reports is that anyone can access the data and conduct their own analysis. All data is also stored on CD's for later access if required.

3.4.2 Analysis Reports

Analysis results obviously form the key part of any trial. The extent, and type, of analysis clearly is dependent on the type of trial (see section 5) but will generally involve an assessment of the change in deflection, rutting and other data over time – either in the absolute sense or as a basis for comparison with the results of other experiments for the purposes of ranking the performance of a pavement or an individual material (e.g. asphalt).

The data collected during ALF trafficking is not analysed in isolation, but considered in tandem with other data such as the results of field and laboratory testing of the pavement materials. Back-calculation, using the back-calculation program EFROMD2, forms an inherent part of many trials because of the need to define the elastic characteristics of the materials in order that performance can be predicted and compared with observed performance.

3.5 Conduct of ALF Trials

Each trial is managed by a Steering Committee, or project Steering Group, composed of representatives of the interested parties. The management of the trials, and their funding, has, however, changed over the last 15 years. Initially, each trial was funded by NAASRA/Austroads and typical funding was A$800,000 per annum. In addition, the 'host' State road agency would provide the test site and facilities, including support staff, at no direct cost to the project. ARRB TR was responsible for the maintenance of ALF, overall management of the site and the collection, processing, analysis and reporting of the data. Members of the trial Steering Committee therefore represented the host road agency and NAASRA/Austroads through the Austroads Pavement Reference Group, which is a reference group made up of senior engineers in all the State road agencies.

By the early 1990s, however, the management and funding mechanisms had changed to reflect the changes that had taken place in the road agencies, particularly their shift in emphasis from a 'road' authority to a 'transport' authority operating 'off budget', and the reduction in funding available for research and development in Australia generally, particularly with respect to pavement and materials technology. In addition, State transport authorities no longer provided the test site and facilities and support staff at no cost to the project.

As a result, whilst Austroads remains the major client, many trials are now funding by 'consortiums' representing Austroads, individual State transport authorities, other Government agencies, private sector utilities and industry, particularly the asphalt and cement/concrete industries, and the membership of the Steering Committees reflects these changes. Associated with the new funding arrangements has been the tendency to conduct trials at 'semi-permanent' sites and, ideally, in Melbourne (where ARRB TR is located) in order to minimise the costs associated with moving ALF interstate, travel and accommodation, etc.

4. Summary of the CAPTIF Research Programme

The research projects conducted at CAPTIF since 1986 are summarised in table 4.1 and a summary of the trials, including relevant research and operational parameters, now follows.

4.1 Inaugural Project

The purpose of this project was to: (1) commission the SLAVE and evaluate its capabilities, and (2) monitor the performance of four granular pavements in order to provide an initial evaluation of the construction and operation techniques required for the accelerated trafficking facility. The average California Bearing Ratio (CBR) of the clayey loess subgrade was 30%. The pavement thicknesses ranged from 100 to 250 mm, in 50 mm increments and, in all four cases, the base material was a well-graded aggregate with a maximum particle size of 40 mm, except for the uppermost lift, which had a maximum size of 18 mm. The surfacing was a double seal coat. Soon after loading began, the seal began flushing, even though there was no loss of chip and the basecourse was firm. After the flushing became severe, the initial coats were removed and the basecourse was lightly re-levelled. A single coat of bitumen, sprayed at a lower-than-normal rate, was coated with a first layer of larger stone chips (ALD of 12 mm) interlocked with smaller chips (ALD of 8 mm).

4.1.1 Research Parameters

Each SLAVE vehicle applied a wheel load of 40 kN, to represent an Equivalent Single Axle (ESA). SLAVE applied 1.53 million ESAs to the pavement during the project. There was no significant difference in the performance of the four pavements, with vertical deformation (rut depth) not dependent on the thickness of the granular cover over the subgrade. Even the thinnest unbound granular pavement of compacted, well-graded crushed aggregate sustained 1.53×10^6 80 kN axle load repetitions in the absence of deleterious ground moisture and environmental factors (Pidwerbesky 1989).

The subgrade strength seemed to increase during the project, probably due to the ideal testing conditions; there was no water infiltration and the subgrade consolidated. The performance of the pavement was better than predicted, primarily because the subgrade strength increased and because there was no water migrating into the granular layer, except in one area of localised failure.

Flushing developed in the chip seal surfacings much earlier than expected. No definite explanation is apparent, but the most likely causes were that the bitumen application rate was too high and the constant heavy wheel load with a relatively short periodicity contributed to the flushing.

Table 4.1 Summary of Projects at CAPTIF

Trial No.	Project and Variables	Load Reps	Wheel Load (kN)/ Tyres Used		Time to: (in days)			
			Veh. A	Veh. B	Const.	Test	Load	Post Mort
C1	Inaugural: 4 thicknesses unbound granular pavements under chip seals	1.53×10^6	40 dual	40 dual	90	370	45	50
C2	Comparative rutting: duals and wide-base single tyres	94,700	40 dual	40 single	30	5	2	20
C3	Effect of particle shape and gradation on basecourse performance: 9 pavements	54,300	40 dual	40 dual	40	20	10	10
C4	Lime-stabilised sub-bases: 3 thicknesses	30,500	21, 40 dual	21, 40 dual	50	40	8	15
C5	Strain response of subgrades and unbound granular pavements: wheel load, tyre pressure and tyre type	51,000	40 dual	21-46 dual	35	85	5	6
C6	Modified binders in asphalt mixes: 6 modified binders	2.1×10^6	40-46 dual	40-46 dual	60	150	60	60
C7	Life-cycle performance of a thin-surfaced unbound granular pavement	740,000	40 dual	40 dual	40	60	30	30
C8	Dynamic wheel loads and pavement wear: single unit and multi-leaf spring suspensions	35,000	38 single	38 single	31	90	3	15
C9	DIVINE[a] (Element 1): air bag and multi-leaf suspensions	1.7×10^6	50 single	50 single	44	220	120	84
C10	Dynamic wheel loads, test 3 air bag and multi-leaf suspensions	250,000	50 single	50 single	35	10	16	10
C11	Subgrade Study	294,000	40 dual	40 dual	40	6	30	10
C12	Cross-test (See section 8.2)	600,000	40 dual	40 dual	45	16	26	10
C13	Comparison of 8 t and 10 t axle loads (commenced 10/99)	1,000,000	40 dual	49 dual	80	20		

[a] Dynamic Interaction of Vehicles and the Infrastructure Experiment.

4.1.2 Operational Parameters

Spraying bitumen by hand wand is unsuitable because the application rate cannot be accurately controlled. Regular sprayers can be used inside CAPTIF, but achieving a uniform application rate on a circular track requires a modified spray bar. A modified design was subsequently developed, but is yet to be implemented.

After some initial debugging and calibration of trigger values, all safety sensors worked properly so that only minimal supervision was necessary, and SLAVE could be left safely operating 24 hours per day, every day of the week. Staff were normally on site only during regular working hours. When safety monitors detected trigger values, SLAVE was stopped until staff arrived to inspect the facility and determine the cause.

Incorporating a typical truck axle and a leaf suspension, the standard CAPTIF vehicles were designed to produce realistic dynamic wheel responses to the changing roughness of the pavement surfaces. The effect of roughness in one segment of the track induces bounce in subsequent segments. Thus, transition zones between test segments are required.

The effects of travelling on a circular track are compensated for by adjusting the wheels' slip angle to a maximum of $\pm3^{\circ}$ offset from the tangent. Thus, the influence on the pavement of cornering can also be investigated. During the first 1 million loading cycles, excessive scuffing developed in the tyre treads, with one inner tyre worn almost to the belts. The scuffing was roughest in the direction of outer to inner tyre tread edge to the crown of the tread, on both tyres in each vehicle. This indicated that the radial component of the forces developed in the tyre-pavement interface were significant. Subsequently, the slip angle was adjusted to $+0.5^{\circ}$. In subsequent projects, the tyre tread depth was measured, and the wheels' slip angles were adjusted by trial and error, to decrease the tyre-scuffing effect of radial acceleration, taking into account the settling of the suspension springs.

When the initial two-coat seal was replaced, some of the basecourse was removed along with the seal, resulting in the new surface being 50 mm lower than the previous surface. The base elevation of the SLAVE had to be lowered in order to level the arms and maintain the dynamic balance of the vehicles and tyres.

Suspension lifter adjustments were necessary to compensate for sagging of the suspension due to wear and extra load in order to maintain the tyres in a vertical plane. When trying to adjust the tyres so that they are vertical, suspensions must be adjusted by turning hand levellers. This adjustment is time-consuming and tedious.

Tractive forces at the tyre/road interface cannot be measured because only the total power input to SLAVE can be read, and this is the total tractive power plus auxiliaries and efficiency losses. The power consumed by each vehicle cannot be requested. A device for measuring the power consumption at each driven wheel should therefore be developed.

The vehicles achieved speeds of up to 50 km/h on freshly laid surfaces, but as the surface condition deteriorated (and the rolling resistance increased), the maximum speed reduced to 44 km/h.

Wheel tracking positions were controlled and reported to within 10 mm of actual position, across the full 1 m width of transverse movement. In the original operating software, the wheelpath position was varied by a random number generator subroutine, to achieve a Gaussian or any other desired frequency distributions. However, the actual distribution never matched the desired distribution because of vehicle stops and the true randomness of the positions. Also, the original software did not provide for recording the accumulation of loading cycles. A better method of achieving the desired overall frequency distribution for a testing routine is by programming a sequence of controlled wheelpath positions and transverse movements. The desired frequency distribution is completed every specified increment (e.g. every 1200 revolutions). The accumulation of these incremental subroutines provides the overall frequency distribution required for a particular project. The stop-start performance of the vehicles would affect the frequency distribution to only a very minor degree.

The experiences of this commissioning project led to the development of new operating software. Bugs and inappropriate logical functions in the original on-shore operating program, as distinct from SLAVE's internal software, were discovered. The experiences indicated the format and characteristics needed in the improved operating program.

This commissioning project exposed the inadequacies of the pavement response-measuring and data-acquisition systems; the Benkelman Beam rebound and surface profile measuring equipment did not provide the accuracy and precision required. As well, there was no instrumentation for measuring strains, stresses, displacements, and moisture condition within the pavement layers.

Consequently, a general purpose, microprocessor-based data-acquisition unit was acquired that connected to the operations computer and a variety of pavement response monitors. An automated profile-measuring device was made that is a modified design of Auckland University's prototype profilometer. Electronic systems to automatically measure strains and displacements, and temperature were acquired.

4.2 Pavement Rehabilitation

The objective of this project was to construct a granular pavement over a weak subgrade, traffic the pavement to failure, rehabilitate the pavement and then traffic the pavement to failure again (Pidwerbesky and Dawe 1990*a*).

4.2.1 Research Parameters

Two attempts were made to weaken the pavement structure from the previous project. In the first attempt, an attempt was made to increase the moisture content of the Templeton soil subgrade by flooding the pavement tank from the surface. However, after three weeks, the water had only penetrated 30 mm into the subgrade. In the second attempt, the pavement tank was flooded from the bottom utilising a drainage layer at the bottom of the subgrade. After three weeks, all the water had travelled up between the concrete pavement tank and the impermeable block of subgrade material.

The third and final attempt at producing a weak subgrade involved removing the granular material and scarifying the top 250 mm of the subgrade. A measured amount of water was added in order to increase the water content of the material to the desired level. This method produced a very weak subgrade: a human walking on the surface would create depressions 50 mm deep. A Dynamic Cone Penetrometer (DCP) would sink through the top 150 mm under its own weight, but below that, the inferred CBR ranged from 10 to over 50. A geotextile was placed over the top of the subgrade, and the basecourse material was placed over that. Because of the weak subgrade, the granular material was unable to be compacted to provide sufficient strength to apply a bituminous surfacing to support the SLAVE. A Benkelman Beam truck completed 16 laps before the pavement became impassable.

4.2.2 Operational Parameters

Two local subgrade materials used at CAPTIF have material properties that make them very sensitive to changes in moisture content. However, they both have low permeability coefficients when compacted, making post-construction changes in moisture content impossible. It is difficult to properly compact basecourse material when it is located on a weak subgrade. The use of a woven geotextile appeared to increase the strength of the pavement, but the pavement deformations were so large that the geotextile tore due to excessive tensile forces.

4.3 Comparative Rutting of Tyre Types

In this experiment, the vertical deformation caused by a single low-profile radial tyre (14.00/80 R 20 on Vehicle B) and dual standard radial tyres (10.00 R 20 on Vehicle A) were compared. The load on each wheel set was 40 kN. The clayey loess subgrade material had an average CBR of 30%. The pavement consisted of a 40 mm thick surfacing of an open-graded bituminous mix, a 150 mm thick basecourse of a high-quality crushed aggregate and a 150 mm thick sub-base of coarse aggregate with a maximum particle size of 65 mm (Pidwerbesky and Dawe 1990*b*).

4.3.1 Research Parameters

After 16,000 loading cycles, the average permanent deformation (as measured by the transverse profilometer) created by the single low profile radial tyre was 92% greater than that of the dual radial tyres.

4.3.2 Operational Parameters

The SLAVE wheelpath wander was set to zero to maximise wheelpath separation of the two vehicles, and the vehicle maintained the same position for the entire loading routine. Modified rims were installed to accommodate the wide-base tyres. Open-graded friction course was successfully employed as a non-structural layer that provided a smooth surface.

4.4 Effect of Particle Shape and Gradation on Unbound Basecourse Performance

In this study, the effect of particle shape and gradation on the performance of unbound basecourse aggregates constructed according to a revised specification (NRB B/2, 1987*a*) was studied. Aggregates consisting of different combinations of rounded (30%, 50% and 70%) and angular, crushed particles were created, for three different particle size distributions using Talbot's equation:

$$P_d = 100 \left[\frac{d}{D}\right]^n$$

where P_d is the percentage of the sample passing a sieve size, d;

d is the sieve size (mm);

D is the largest particle size in the sample (mm); and

n is the gradation exponent.

The values for the gradation exponent (n), 0.4, 0.5 and 0.6, represent the lower limit, mid-point and upper limit of the gradation envelope for New Zealand primary basecourse aggregate, respectively (NRB M/4, 1987*b*). A total of nine basecourse aggregates were created. The basecourse aggregates were placed in nine sequential segments in the track, with an average depth of 108 mm ± 7 mm; the maximum dry density varied according to the gradation. Segments A, B, C and I could not be compacted, so they were removed and replaced with a local aggregate; the four segments, which were adjoining, were combined into one segment designated as A1. The 48 mm (± 6 mm) thick surfacing was an open-graded bituminous mix (porous asphalt or friction course) (Pidwerbesky *et al.* 1990).

4.4.1 Research Parameters

After 54,000 ESAs cumulative loadings, the subgrade deformation under loading was similar for all test segments, while the basecourse deformation differed. Particle shape had the greatest effect on the performance of the aggregates, compared with gradation. Aggregates consisting of 30% or less angular particles could not be compacted, and the best performance was achieved with aggregates of 70% or more angular particles, which is required by the New Zealand basecourse aggregate specification.

4.4.2 Operational Parameters

Nine segments were tested in the track. The length (6 m) of each segment was too short because the borders of each segment must be sacrificed for a transition zone between adjacent segments.

Achieving the desired particle size distributions in the aggregates was extremely difficult using the normal crushing and screening processes of a quarry, and none of the aggregates completely satisfied the specified gradations.

The new CAPTIF Profilometer for measuring transverse profiles was commissioned. The device was accurate and precise, but the print-out had to be manually interpreted, so the device was subsequently modified to provide a digital output.

4.5 Behaviour of Lime-Modified Sub-Bases

In this experiment, three pavements were constructed, two with lime-stabilised clay sub-bases of thicknesses 150 mm and 250 mm, and the third with an unmodified high quality, well-graded, crushed aggregate. The laboratory CBR of the unstabilised and stabilised clay specimens was 5% and 20%, respectively. For all three pavements, the surfacing was a 30 mm thick layer of asphalt and the basecourse was a 150 mm thick layer of high quality, well-graded crushed aggregate (NRB M/4, 1987b). The subgrade had an unsoaked CBR of 3%, which represented the worst possible case, and a compacted dry density of 1.7 t/m^3 ± 4% at a moisture content of 20%. Laboratory tests showed that the optimum lime content for the sub-base material was 4%. The maximum dry densities (at optimum moisture contents) of the unstabilised and lime-stabilised sub-base material were 1.68 t/m^3 (at 18%) and 1.52 t/m^3 (at 25%), respectively. A geotextile (Typar 3407) was placed on top of the sub-base to separate the basecourse aggregate material and the sub-base, therefore enhancing the measurement of the layer profiles without interfering with the stress development and distribution within the pavement. The minimum temperature during curing was +6 °C; during loading, the pavement temperature ranged between –3°C and +20°C.

Elastic deflections and permanent deformation of the pavement surface were measured. Pavement failure was defined as vertical surface deformation of 25 mm. The pavement containing the 150 mm thick lime-stabilised layer performed substantially better than the same thickness of unstabilised aggregate. Increasing the stabilised sub-base thickness by 100 mm yielded a fifteen-fold increase in the life of the pavement. The moduli of the lime-stabilised layers were lower than that predicted by laboratory testing and computer analyses, primarily because the pavement could not be fully compacted on such a weak subgrade (Owiro and Pidwerbesky 1990).

4.6 Fundamental Behaviour of Unbound Granular Pavements Under Various Loading Conditions

The purpose of this study was to examine specific fundamental loading parameters (load magnitude and number of repetitions, tyre inflation pressure and basic tyre type) that influence the behaviour of thin-surfaced granular pavements. The pavement response and performance measurements included continuous surface deflection basins, longitudinal and transverse profiles, and vertical strains in the granular layers and subgrade (Pidwerbesky 1996). The stage II pavement was part of a commercial test in which the performance of various asphalt mixes were tested

4.6.1 Stages of project

4.6.1.1 *Stage I – Pavement response to tyre type, inflation pressure and wheel load*

The first pavement trial considered only the elastic response of a thin-surfaced, unbound granular pavement over a weak subgrade, to varying wheel loads, tyre inflation pressures and two basic tyre types (bias and radial ply). Vehicle A carried a constant half-axle load of 40 kN (equating to a full axle load of 80 kN) with dual bias ply tyres inflated to 550 kPa, so that it was a reference throughout the testing routine. The characteristics of vehicle B were modified. The maximum cold tyre inflation pressures allowed by the tyre supplier were 700 and 825 kPa for the bias and radial ply, respectively. All radial and bias ply tyres were 10.00R20 and 10.00x20, respectively. The dynamic characteristics of each vehicle were evaluated to confirm that they were similar. Dynamic wheel forces were measured at a constant speed of 40 km/h.

4.6.1.2 *Stage II – Effect of binder modification on asphalt pavement performance*

This trial involved constructing six test sections of various asphalt mixes over 200 mm of unbound granular basecourse and a silty clay subgrade possessing an in-situ CBR of 13%. The basecourse aggregate was a well-graded, crushed gravel, compacted at a moisture content of 4% to a maximum dry density of 2.15 t/m^3. The design life of all test sections was 1.0 x 10^6 ESA, so the depth of the asphaltic concrete varied from 80 mm to 125 mm, depending on the characteristics of the different mixes. The bitumens used for the test sections were:

1. a standard paving grade (conforming to a German specification for B80 Grade,
2. a binder modified with a plastomeric polymer,
3. binders modified with three types of elastomeric polymer, and
4. a high-stiffness (pen. grade 21 @ 25°C) binder.

The wheel load was 40 kN for both vehicles for the first 920,000 loading cycles, and 46 kN for the remaining 1.2 million loading cycles. The dual radial tyres in both vehicles were inflated to 700 kPa, and the vehicle speed was 40 km/h. Altogether, SLAVE applied 3.2 million ESA to the test pavements. Details of the project and results are provided in Stock *et al.* (1992). The rut depth was only 4 mm, indicating negligible compaction in the sub-surface layers. The project concluded before the pre-defined failure criterion of a maximum surface rut depth of 25 mm occurred because the pavement design was conservative (pavements designed for 1 x 10^6 ESA should have exhibited greater deterioration after 3.2 x 10^6 ESA) and because the project costs exceeded the budgeted funds. The test sections exhibited negligible deterioration in their structural condition and minimal surface distress. Stock *et al.* (1992) concluded that the thinner asphalt layers constructed with modified binders and the high-stiffness binder provided performance equivalent to that of the thicker layer containing a conventional binder.

4.6.1.3 *Stage III - Life-cycle performance of a thin-surfaced unbound granular pavement*

The test pavement for this trial consisted of 25 mm of asphalt surfacing over 135 mm thick basecourse of unbound granular aggregate on a silty clay subgrade of CBR 13%; the material properties were the same as those described above. The pavement responses and properties were measured as described above. The pavement was subjected to a constant loading condition (40 kN load and dual radial tyres inflated to 825 kPa) until the loading concluded at 740,000 ESAs.

4.6.2 Research parameters

CAPTIF was utilised to investigate the fundamental behaviour of subgrades and unbound granular pavements under various loading conditions. An electronic-based data-acquisition system for accurately measuring strains in unbound granular layers and subgrades has been developed and employed successfully in a number of projects. Instead of relying on simplistic relationships between static axle loads and performance, fundamental pavement responses can be measured for input to pavement performance prediction models. Any procedures for determining load equivalency factors must also consider the type of pavement and the bearing capacity of the subgrade (Pidwerbesky 1996).

With respect to pavement and subgrade response to loading, and for the specific conditions of the investigation, the tyre type (10.00R20 radial and 10.00x20 bias ply) had an insignificant effect. The axle load had the greatest effect on pavement response, but increases in the tyre pressure (between 550 kPa and 825 kPa) resulted in slight decreases in the magnitude of the vertical compressive strain in the subgrade and unbound granular cover.

The magnitude of the vertical compressive strain in the subgrade increased initially, then remained relatively constant. The vertical compressive strain in the unbound basecourse aggregate tended to decrease slightly in magnitude under cumulative loading; the basecourse aggregate compacted under repetitive loading, then reached a stable condition. The relationship between vertical compressive strains and the cumulative loadings became stable after the pavement was compacted under initial trafficking (in the absence of adverse environmental effects).

The strain magnitudes measured were greater than the levels permitted by the four subgrade strain criteria evaluated; the criteria are intended to govern the allowable vertical compressive strain in the subgrade, so as to ultimately limit pavement rutting. The subgrade strain criteria are conservative, but the Austroads criteria were the closest to the actual results.

4.6.3 Operational parameters

Thin layers of asphalt (30 to 40 mm) were successfully placed and used as a thin surfacing for testing unbound granular pavements.

Accelerometers were attached to the axles and chassis of the SLAVE vehicles, and vertical accelerations of those unsprung and sprung masses were measured.

The DIPStick Profiler was proven to be an accurate and repeatable, though laborious, means of measuring longitudinal profiles of the test pavement, for determining roughness. The advantage of the circular track was that closure of the level survey provided an accurate measure of the error in measurements, which was found to be a systematic cumulative error. The DIPStick data was processed and the error compensated for by distributing the error and adjusting each measurement point.

Six segments were tested simultaneously in one project.

Procedures were developed for using a Falling Weight Deflectometer (FWD) inside CAPTIF, for marking and photographing cracks with appropriate lighting, and for drying the subgrade material uniformly. However, an enclosed storage area is required for maintaining stockpiles of materials at the appropriate moisture conditions.

4.7 Dynamic Wheel Forces and Pavement Wear

The objective of this research programme (1992-1998) was to compare the pavement deterioration caused by dynamic loads generated under different types of suspensions: steel parabolic leaf spring and shock absorber, multi-leaf steel suspension, and air bag suspension with shock absorber. Using the accelerometers and displacement transducers fitted to the SLAVE vehicles, vertical dynamic loads created by the vehicle bounce were related to sub-surface strains and longitudinal surface profiles measured by both the DIPStick profiler and a laser device mounted on one vehicle. Three pavements were tested as part of this programme (de Pont *et al* 1999).

4.7.1 Operational parameters

Because additional pavements were to be constructed and tested for dynamic loading effects, sufficient subgrade soil and basecourse aggregate were procured for five pavements and were stockpiled, to ensure that the material properties of the pavement were the same for each suspension. The basecourse aggregate of well-graded, crushed gravel was produced to stringent specifications (NRB M/4, 1987b) using a portable aggregate blending plant. Before the commencement of these tests, SLAVE was modified to allow three different types of suspension systems to be fitted to the vehicles. The suspension systems were: multi leaf trapezoidal leaf spring with no viscous damping, single or double leaf parabolic leaf spring with viscous damping, and dual air bag with viscous damping. Each test was run with the two vehicles offset by 800 mm in order to provide the maximum separation between the two wheelpaths. This allowed two suspension systems to be tested simultaneously on identical pavement structures.

4.7.2 Details of Test 1 – Research parameters

4.7.2.1 *Research parameters*

The test pavement for this trial consisted of 25 mm asphalt surfacing over a 250 mm thick basecourse of unbound granular aggregate on a silty clay subgrade of CBR 13%. The vehicle configurations were steel multi-leaf spring and double parabolic spring. The vehicles were loaded to 37 kN and a 385/65R22.5 wide base single tyre was fitted to the vehicles.

4.7.2.2 *Operational parameters*

The pavement was subjected to 35,000 load applications before the deteriorating pavement halted loading. A post-mortem of the pavement revealed that the damage was confined to the basecourse layer and was primarily due to insufficient compaction during construction. The rate of deterioration was enhanced by the suspensions only being loaded to 60% of their rated capacity and, as a result of this, the damping systems were unable to provide enough damping to prevent damage around the track at locations corresponding to the natural frequency of the suspensions.

4.7.2.3 *Conclusions*

The results of this test showed the need for proper compaction of the basecourse material: a tandem steel-drum roller can compact the material in the vertical direction, but the width of the drum and the constant turning of the roller on the circular tank kept disturbing the top layers of the basecourse. As a result of this, satisfactory compaction of the basecourse was unable to be obtained using this method of compaction. After the conclusion of the test, trials were conducted with smaller tandem-drum rollers and large plate compactors and consequently a large vibratory plate compactor was purchased for use at CAPTIF. The performance of the pavement and vehicles showed that, while a suspension system can be designed to be road friendly, if the static axle loads are not close to the design load of the suspension system, then the external damping will not provide enough damping and the suspension system will not behave in a road friendly manner.

4.7.3 Details of Test 2

4.7.3.1 *Research parameters*

This test was part of the OECD DIVINE experiment (OECD 1997). The pavement consisted of 85 mm of asphalt (AC 16 mix) over 200 mm of granular basecourse and 1220 mm of silty-clay subgrade having a CBR of 12%. The vehicle configurations were steel multi-leaf spring and dual air bag. The vehicles were loaded to 49 kN and a 385/65R22.5 wide base single tyre was fitted to both vehicles.

4.7.3.2 *Operational parameters*

The pavement was subjected to 1.7 million load applications before loading stopped due to budget constraints. At the conclusion of loading, neither of the wheelpaths had reached the pre-defined failure conditions. Due to the tight time constraints and high rate of loading, systems were developed to enable the historical trends of the different data types to be available two days after the measurement of the data. A system for measuring surface cracking was developed based on the system used by ARRB TR staff (see section 3.3). ARRB TR H-bar strain gauges and Partial Deflection Gauges (PDGs) were installed in the pavement. An ARRB TR single laser profiler was installed part way through the project.

4.7.3.3 *Conclusions*

The pavement instrumentation survived the high rate and number of loadings, with the exception of the H-bars. All of the H-bars survived the pavement construction, but they had all failed after 250,000 load applications. One possible reason for this is that the gauges were subjected to high strains (up to 900 $\mu\varepsilon$), which was beyond the mechanical strength of the carrier bars and cable jointing. Of the 34 Bison coil pairs installed in the pavement, 26 performed correctly until the end of loading. This was the heaviest sustained loading that the strain coil system had been subjected to at CAPTIF thus far.

Once again, SLAVE proved its ability to operate continuously for long periods of time, imparting realistic dynamic loads to the pavement at a high rate.

The simultaneous testing of two vehicle configurations, each travelling in their own wheelpath, proved successful.

Thickness design for test pavements must make allowance for the favourable operating environment at CAPTIF in order to meet failure criteria within the scheduled project duration.

4.7.4 Details of Test 3

4.7.4.1 *Research parameters*

The test pavement for this trial consisted of 25 mm asphalt surfacing over a 250 mm thick basecourse of unbound granular aggregate on a silty clay subgrade of CBR 13%. The vehicle configurations were steel multi-leaf spring and dual air bag. The vehicles were loaded to 49 kN and a 385/65R22.5 wide base single tyre was fitted to both vehicles.

4.7.4.2 *Operational parameters*

The pavement was subjected to 300,000 load applications before loading was stopped due to budget/contractual limits being reached. At the completion of loading the pavement had failed to reach the predetermined failure levels, but the performance of the pavement was similar to Test 2.

4.7.4.3 *Conclusions*

The dynamic wheelforces generated by the two different suspension systems were substantially different, but the mean level of pavement deformation was approximately equal for both suspension types. However the variation in the deterioration was much greater for the steel suspension than for the air suspension. A moderate correlation was found between the change in pavement profile, dynamic wheelforces, and initial pavement strength.

4.8 Materials and Methods to Prepare Subgrades Suitable for CAPTIF

4.8.1 Research parameters

A suitable material and/or construction method was required to produce a weak subgrade (CBR < 5%) for use at CAPTIF. A suitable material was found that exhibited different strengths based on the water content of the material, when it was compacted as close as possible to zero air voids. Sufficient material was procured and shipped to the CAPTIF site. The material was altered by the addition of water to a material with three separate moisture contents. The aim of the test was to see if the pavement could be constructed with different water contents in the subgrade material, and to see if the material properties varied over the loading phase of the project. The project is described in Arnold, Bartley Consultants and Steven (1999).

4.8.2 Operational parameters

The water content of the subgrade material was altered by using soak hoses over the material, and then using a tractor mounted rotary hoe to mix the additional water and clay to a uniform material. A test pavement was successfully constructed on the weak subgrade and 294,000 load applications of a standard 40 kN axle were applied.

4.8.3 Conclusions

The water content and dry density of the subgrade material did not alter significantly from the start to finish of the loading sequence. This showed that controlling the water content of the material was a valid way to control the strength of the subgrade, however improvements need to be made in the methods used to alter the water content of the material and in the techniques used to mix the material to a uniform state.

4.9 CAPTIF/ALF Cross Trial

See section 8.2 for details.

5. Summary of ALF Trials

A summary of all the trials conducted to date is presented in table 5.1. It can be seen from the table that the 25th trial was in progress and that, since trafficking commenced in 1984, almost 25,000,000 cycles have been applied in over 200 experiments to about 120 test pavements. The number of cycles is equivalent, assuming the 'fourth power law' to about 2.7 x 10^8 ESAs.

A perusal of the types of trials suggests that ALF has been used to address three broad issues:

- proof testing of in-service pavements,
- design and construction procedures, and
- alternative materials and practices.

In recent years, the main thrust of the ALF programme has been directed to the last of these issues, particularly the formulation of guidelines for the most efficient use of alternative materials and construction procedures in terms, if possible, of current design procedures, especially those contained in the Austroads (1992) Pavement Design Guide. More recently, 'asset management' issues such as 'maintenance intervention practices' are being addressed and this issue is expected to gain more prominence in future years as transport authorities assume a far greater 'policy' role.

The change in emphasis also reflects, the fact that the funding of trials has changed. Although Austroads has remained the major client, substantial financial support has been forthcoming from individual State Transport Authorities, Government and private agencies, and industry associations and private industry, particularly oil companies. "In-kind" support, in the form of the provision of pavement materials, pavement construction and laboratory testing, etc. has also been provided.

Some details of the trials, highlighting the major findings, follow. The trials are summarised in more detail in Sharp, Johnson-Clarke and Vuong (1998), which includes a list of all the documents produced under the ALF programme.

5.1 Commissioning Trials of Unbound Pavements

5.1.1 Somersby, NSW

The first trial was conducted between July 1984 and April 1985 on a heavy duty unbound pavement with an asphalt surface. The major aims were to proof trial ALF and evaluate the instrumentation used to record pavement response, examine the performance of an existing heavy duty pavement, and to derive load-damage relationships.

The trial verified that a single-sized crushed stone macadam sub-base could be used with confidence in heavy traffic applications.

The trial also confirmed that ALF had the potential to be a very useful tool for the evaluation of pavement performance.

Table 5.1 Summary of ALF Trials

Trial No.	Type of Trial	Location	Clients	Duration	No. of Pavement Types	No. of Expts	No. of Load Cycles			
							40 kN	50 kN	60 kN	80 kN
A1	heavy duty unbound pavement with thin asphalt surface (proof test)	Somersby, NSW	Austroads; RTA, NSW	07/84-04/85	1	1	43,912			718,640
A2	heavy duty unbound pavement with chip seal surface (proof test)	Benalla, Vic.	Austroads; VicRoads	06/85-02/86	1	1	16,044		15,690	1,369,248
A3	cement-treated bases & sub-bases	Beerburrum, Qld	Austroads; DMR, Qld	02/86-02/87	6	19	496,304		78,577	1,632,441
A4	unbound & stabilised blast furnace slag	Prospect, NSW	Austroads; RTA, NSW	07/87-05/88	5	19	860,536		352,362	843,857
A5	asphalt rehabilitation treatments	Callington, SA	Austroads; Transport, SA	07/88-10/89	14	17	389		530,972	2,791,208
A6	fatigue properties of asphalt & cement-treated crushed rock	Mulgrave, Vic	Austroads; VicRoads	11/89-03/91	4	21	189,971		1,372,631	1,638,891
A7	geotextile reinforced seals	Brewarrina, NSW	Austroads; RTA, NSW Brewarrina Shire Council	07/91-12/91	5	12	927,165	89,640		
A8	fine grained marginal materials	Beerburrum, Qld	Austroads; DMR, Qld	02/92-06/93	7	14	313,000	150	689,840	94,800
A9	bitumen & bitumen/cement-stabilised crushed rock	Beerburrum, Qld	Austroads; DMR, Qld	02/92-06/93	5	8			384,900	
A10	crushed rock axle load equivalency	Beerburrum, Qld	Austroads; DMR, Qld	02/92-06/93	1	4	252,100		413,800	239,000
A11	crushed rock moisture/compaction	Beerburrum, Qld	Austroads; DMR, Qld	02/92-06/93	6	8		418,200		212,700
A12	lateritic gravel bases & sub-bases	Beerburrum, Qld	WES; LTRC; Austroads; DMR, Qld	06/93-03/94	5	9		173,268	144,369	613,536
A13	rut-resistant properties of asphalt: I	Beerburrum, Qld	Austroads; AAPA	11/93-03/94	10	8				878,171
A14	deep lift insitu recycling	Cooma, NSW	Austroads; RTA, NSW	05/94-10/94	4	5	46,622	47,571	56,683	760,062
A13	pavement heating system	Beerburrum, Qld	Austroads; AAPA	10/94-01/95	6	2				109,009
A15	rut-resistant properties of asphalt: II	Beerburrum, Qld	Austroads; AAPA; Shell; Mobil; BP	01/95-04/95	15	17		67,400		575,292
A16	stabilised flyash	Eraring, NSW	Pacific Power	05/95-12/95	6	17	164,643	36,380	125,491	541,394
A17	crushed rock axle load equivalency	Dandenong, Vic.	Austroads; VicRoads; Boral	03/96-12/96	2	8	90,164	115,651	1,700	

Table 4(con't)

Trial No.	Type of Trial	Location	Clients	Duration	No. of Pavement Types	No. of Expts	No. of Load Cycles			
							40 kN	50 kN	60 kN	80 kN
A18	insitu stabilisation of marginal sandstone	Dandenong, Vic.	Austroads; AustStab; AAPA; Pioneer; Mobil; DMR, Qld; Shell; Transport, SA; VicRoads; ICL; Astec; Pave. Tech.; Boral	05/96-08/96 01/97-03/97	4	11		300,956	1,700	342,548
A19	fatigue performance of asphalt: Pilot Study I & II	Dandenong, Vic.	Austroads; AAPA	09/96-01/97 04/97-07/97	2	3				543,284 (+ 161,487 cycles @ 90 kN)
A20	granite sett pavements	Dandenong, Vic.	Sydney Opera House Trust; SKM	07/97-10/97	5	3	79,264		15,770	110,108
A21	fatigue performance of asphalt: III	Dandenong, Vic.	Austroads; AAPA	06/98-09/98	2	1				500,030
A22	maintenance intervention (pilot)	Avenel, Vic.	Austroads	09/98	1	1	13,957			39,914
A23	design characteristics of rigid pavements: Stage I	Goulburn, NSW	Austroads; RTA, NSW; Comm. DTRS; C&CAA; DMR, Qld; VicRoads	10/98-12/98	3	4				302,275
A24	maintenance intervention (stage 2)	Ruffy, Vic.	Austroads	03/99-05/00	3	3	27,027			226,172
A23	design characteristics of rigid pavements: Stage II	Goulburn, NSW	Austroads; RTA, NSW; Comm. DTRS; C&CAA; DMR, Qld; VicRoads	11/99-03/00	5	5				643,509
A25	maintenance intervention (stage 3)	Dandenong, Vic.	Austroads	06/00-						
	Total				123	216	3,521,098	1,249,216	4,184,485	15,726,089(+ 161,487 @ 90 kN)
	Total Cycles	*24,842,295*								
	*Total ESAs**	*272,543,129**								

* Assuming 'fourth power law'.

5.1.2 Benalla, Victoria

The second trial was conducted between June 1985 and February 1986 on a new section of the Hume Highway at Benalla just before it was opened to traffic. The surface was composed of a double seal rather than asphalt.

Performance was excellent, with little structural deterioration observed. However, frequent patching of the surface seal was required, especially after rainfall, reflecting the effect of the tractive forces of the ALF wheels as they met the pavement. Since that time, trials have generally been conducted using a thin asphalt layer as the surface course rather than a chip seal.

5.2 Cement-Treated Crushed Rock Pavements

The main purpose of the third trial, conducted at Beerburrum, Queensland, between February 1985 and February 1986, was to test the performance under load of "thin" 200 mm and "standard" 300 mm thick cement-treated crushed rock (CTCR) pavements which were constructed to similar standards.

The typical failure mode was de-bonding of the layers, followed by erosion at the interfaces. The damage in the pavement layers was found to progress from the upper layers towards the lower layers. The behaviour observed under ALF duplicated that which had been observed on the adjacent National highway, which led to an increased confidence in the ability of ALF to duplicate field behaviour. Queensland Department of Main Roads (QDMR) amended their construction procedures in the light of the results.

5.3 Blast Furnace Slag

Changes in the iron-making process had led to changes in the chemical composition of ground granulated blast furnace slag and a reduction in crushing strength. Although similar material had been used in applications overseas, there was no evidence of its use under heavy traffic with only a thin bituminous surface seal.

The main purpose of this trial, which was conducted at Prospect, NSW, between July 1987 and May 1988, was to examine the relative performance of unbound and stabilised blast furnace slag materials.

It was found that both the unbound and stabilised slags could be used in place of unbound and stabilised crushed rock provided they were protected from excessive tensile strains by the provision of an adequate subgrade and that an adequate wearing course, preferably asphalt, was used to prevent surface wear.

5.4 Rehabilitation Treatments Using Asphalt

This was the first trial to examine the performance of a range of asphalt surfacings in the context of pavement rehabilitation. The test strips consisted of a series of thin overlay treatments and a series of asphalt reinstatements.

The trial, which was conducted at Callington, South Australia, between July 1988 and October 1989, pointed to the high sensitivity of asphalt failure mode to temperature and the relative deformation performance of various binders, including modified binders, under trafficking.

Deformations at high temperatures were mainly confined to the upper asphalt layer, indicating that only this layer needed to be replaced with a more rut-resistant product when repairs due to rutting were carried out. A section incorporating a high bitumen content mix in the lower base did not excessively deform, indicating that these layers could be used in thinner asphalt structures than previously thought necessary and/or that the addition of bitumen will improve fatigue life.

5.5 Fatigue Performance of Bound Materials

The aim of this trial, which was conducted in Melbourne between November 1989 and March 1991, was to improve the methods for predicting the fatigue life of bound materials by determining the fatigue characteristics of asphalt and cement-treated crushed rock (CTCR) and comparing the field behaviour with laboratory predictions.

Relationships were established between the back-calculated asphalt stiffness and CTCR modulus, determined from FWD deflection bowls, pavement temperature and the severity and extent of surface cracking. The asphalt stiffness and CTCR modulus was found to decrease markedly with an increase in the number of loading cycles before surface cracking was apparent.

It was recommended that the relationships developed be used in a revision of the procedures for the characterisation of asphalt and cemented materials then recommended in the Austroads Pavement Thickness Design Guide, that laboratory investigations of the effects of CTCR modulus on fatigue life be undertaken, and that an investigation be conducted to determine, for various classes of road, the degree of asphalt cracking where life cycle costs are lowest.

5.6 Geotextile-Reinforced Seals on Expansive Clay Subgrades

The main purpose of this trial, which was conducted at Brewarrina, NSW, in 1991, was to compare the performance of low cost geotextile reinforced seal pavements placed directly onto a black clay soil subgrade, especially in the situation when water was lapping to the edge of the seal, with that of the gravel pavement normally adopted in these situations.

The major output of the trial was guidelines for the design and management of all-weather low cost pavements involving the use of geotextile reinforced seals over expansive clay subgrades, whilst the major benefit was the more effective use of local materials and scant resources in a location where it is imperative to maintain road links during the "wet season".

Given the wide distribution of black soils throughout Australia, the results of the trial had the potential to be applied extensively throughout Australia. This was the first ALF trial conducted in a remote area of Australia and also the first trial involving a substantial input from local government (Brewarrina Shire Council).

5.7 Fine Grained Marginal Material

The sandstone paving material found beneath the expansive black soil surface horizon in the arid (less than 400 mm mean annual rainfall) area of Western Queensland has quite poor engineering properties compared to standard paving materials, yet extensive QDMR experience over approximately 30 years had indicated that, with appropriate design and maintenance strategies, adequate performance could be achieved. The challenge was to continually refine design, construction, maintenance and rehabilitation practices to ensure optimal performance while minimising whole of life costs.

Pavement deformation was generally confined to the surface seal and the unstabilised/stabilised sandstone layers, suggesting that the stiff clay subgrade played a significant load-carrying role, whilst the unstabilised or stabilised sandstone layers, which did not have a higher operating strength than the subgrade, supported the surface seal and provided little additional load-carrying capacity.

The life of the typical pavement with a nominal 125 mm of unstabilised sandstone exceeded the maximum design life of 10^6 ESAs expected for this pavement type under dry conditions. The results also suggested that the thinner pavements could perform as well as the thicker pavements under low axle loads (40 kN) but that the effects of pavement thickness may be significant under higher axle loads (>60 kN). In addition, the effects of sandstone quality were not significant under low axle loads.

As a result of these trials, design changes were made by QDMR to widening and pavement rehabilitation projects which resulted in savings of approximately 15% of total project costs. The concept was also to be transferred to different climatic regions (up to 500 mm mean annual rainfall) using different non-standard paving materials and subgrades on a trial basis.

5.8 Crushed Rock Moisture/Compaction

The sandstone paving material found beneath the expansive black soil surface horizon in the arid (less than 400 mm mean annual rainfall) area of Western Queensland has quite poor engineering properties compared to standard paving materials, yet extensive QDMR experience over approximately 30 years had indicated that, with appropriate design and maintenance strategies, adequate performance could be achieved. The challenge was to continually refine design, construction, maintenance and rehabilitation practices to ensure optimal performance while minimising whole of life costs.

The low-plasticity, highly-permeable crushed rock bases, when placed on a CTSB having a 3% crossfall, quickly dried back in the dry environment operating at that time at Beerburrum. As a result, the moisture contents were much lower than those immediately after sealing and the pavements were much stronger than expected.

As a result of the trial, recommendations were made regarding the current QDMR specifications for compaction and moisture at sealing, specifically: (1) the control of compaction and moisture content at sealing in order to maximise base performance; (2) the risk of early pavement failure at combinations of high level of compaction and high degree of saturation; and (3) the risk of large errors in OMC and MDD values of low-plasticity crushed rocks determined using the current Standard compaction techniques.

5.9 Lateritic Gravel Pavements

Whilst the US Army Corps of Engineers Waterways Experiment Station (WES) had widespread experience with the use of laterites for the construction of lightly-trafficked road pavements in tropical regions, there had been no controlled loading experiments to compare different qualities of material. Ferricrete (laterite) gravels are also used in pavements in south-west Queensland depending on the economic proximity of sources to the projects. In some cases, however, costly modification of the ferricrete properties is required to correct specification deficiencies (grading and plastic properties) often involving screening and crushing.

Tests were conducted on "good" and "poor" lateritic gravels, constructed to both Standard and Modified compaction standards, and either full depth or over a cement-treated sub-base (CTSB), under contract to the Louisiana State University on behalf of the WES and QDMR. This was the first ALF trial conducted under contract for an overseas client.

The performance of both laterites under dry conditions was excellent and the performance of the higher quality material constructed full depth was similar to that of the pavements constructed on the CTSB. The performance of the higher quality material under wet conditions was better than anticipated, whilst that of the lower quality material was, as expected, poor. The results of the trial have already been widely incorporated into practice, especially in Queensland.

5.10 Rut-Resistant Properties of Asphalt Mixes

A trial was conducted at Beerburrum, Queensland, during 1994 and 1995 to evaluate the relative rut-resistant properties of a range of conventional and modified asphalt mixes and to compare the field performance observed under ALF with that predicted in the laboratory as a means of validating, or otherwise, the use of the dynamic creep test as a mix design tool. Testing was also conducted under contract for three asphalt companies.

Initial testing was hampered by the fact that pavement temperatures varied and were generally much lower than the temperatures at which rutting occurs in Australian pavements. When follow-up testing was conducted at a controlled temperature of 50°C, using a specially-developed pavement heating system, the relative rankings of the mixes were different, with the Multigrade and SBS binders ranking much higher, confirming the trends suggested by the binder testing program.

There was a clear dependence of resilient modulus and minimum creep slope on air void content and the minimum slope continued to decrease below 3% voids, suggesting that the creep test may provide misleading information regarding the field performance of mixes having low air voids. The amount of added pozzolanic filler (material passing the 75 μm sieve) also greatly influenced the results and was a dominant variable controlling the creep slope.

The analysis of the laboratory creep test results suggested that the minimum creep slope could rank the relative performance of mixes having the same composition but different binder type but not the relative performance of mixes having different gradings and compositions, which has implications in terms of its use in an asphalt mix design procedure.

The results of the ALF testing at 50°C correlated with the results of laboratory wheel-tracking testing conducted at 60°C, which suggested that wheel-tracking could be used as a surrogate for ALF results. This could be particularly useful for the design of heavy duty mixes.

5.11 Deep Lift Stabilisation Using Cementitious Binders

The development of modern deep-lift recycling equipment and specialised binder spreaders, together with the more ready availability of a range of slow-setting binders (granulated slag, flyash and lime) and high performance compaction equipment, had enabled recycling to be conducted to much greater depths than previously possible. This had resulted in savings of 20-40% over the cost of conventional granular overlays, or a \$4M-\$6M per annum saving for a \$20M rehabilitation programme.

A trial was carried out at a site near Cooma, NSW, during 1994 to assess the long-term effectiveness of deep-lift recycling. Experiments were conducted on stabilised granular pavements of various depth and an unbound "control" pavement over subgrades of relatively low and relatively high strength. A single stabilising agent of 85% ground granulated slag and 15% hydrated lime, with a target application rate of 5% by mass, was adopted.

It was found that, even with a low strength subgrade, the recycled pavements would perform satisfactorily under moderate rural traffic loadings up to about 10^7 Equivalent Standard Axles (ESAs). The results suggested that for high subgrade strengths the pavements would also provide adequate service at higher traffic loading.

It was concluded that the Austroads cemented materials fatigue relationship generally under-predicted the fatigue life of the trial material but that there was justification in using a conservative approach for the design of in situ stabilised pavements because of the variability of existing materials, thickness, binder quantity, compaction and curing.

5.12 Stabilised Flyash

Pacific Power conducted a research and development project, the aim of which was to demonstrate the cost-effective use of flyash (a waste product from conventional coal-fired power generation) in road construction in order that new markets for large volume flyash application could be established, and/or existing markets enhanced. Part of the project involved an ALF trial at Eraring Power Station during 1995.

The distress mechanisms observed under accelerated loading were different for cement-stabilised flyash base and sub-base pavements. In the case of the cement-stabilised flyash base pavements, the mechanism was fatigue followed by crushing of the material. Where cement-stabilised flyash was used as a sub-base under a granular basecourse, the pavements rutted after a relatively low number of loading cycles, with rutting of the granular base being the principal distress mechanism.

Using "crushing" life performance relationships developed from the data, interim design charts for thick (>200 mm) cement-stabilised flyash base pavements were derived. It was recommended that these charts be revised as additional field performance data become available. In-service field trials are also being conducted by Pacific Power.

5.13 Axle Load Equivalency of Crushed Rock and Marginal Sandstone

Australian reviews of vehicle mass limit regulations were hampered by the lack of relevant performance data on the impact of axle load, tyre type and tyre pressure, especially with respect to chip seal pavements, which formed the bulk of the Australian sealed road network. There was a need to investigate the implications of increased axle loads and related parameters on the pavement design procedures recommended in the Austroads Pavement Design Guide. In addition, there was a need to develop relationships between the properties of unbound materials determined in the laboratory and observed field performance. The trial was conducted during 1996 at a new "semi-permanent" site at Dandenong, Victoria.

Because the effects of moisture on the performance of granular pavements are very significant, it was difficult to compare the results obtained from the different sections. It was recommended that no further axle load equivalency trials using the ALF be attempted until the test sites are protected from the effects of rainfall.

The load damage exponent (LDE) value determined for the crushed rock and marginal sandstone pavement was about 7.5, which was much higher than the LDE of 4 obtained from the AASHO Road Test, but similar to the "subgrade damage" exponent of 7.14 recommended in the Austroads Pavement Design Guide. On the basis of this, albeit limited, data, there is a need to review the load damage factors for flexible pavements in Australia, particularly for residential street pavements consisting of a thin unbound crushed rock base and a low-CBR subgrade.

5.14 Insitu Stabilisation Treatments

The use of pavement rehabilitation treatments and materials has been increasing in Australia but no guidelines are as yet provided in the Austroads Pavement Design Guide as to the most appropriate use of these treatments, including issues such as the effect of curing time on performance, and also the relevance of the current fatigue life prediction models to these materials.

During 1996 and 1997, the performance of a high-quality crushed rock and a marginal material when they are unstabilised and when they are stabilised with bitumen/cement and slag/lime blends was evaluated and the results compared with laboratory data.

The difficulties associated with compacting marginal materials at depth were apparent, and also the fact that the density of the unbound material prior to stabilisation was variable. The lack of curing of the slag/lime section would also have affected the results. It was therefore recommended that a maximum compaction standard of 95% of Modified maximum dry density be adopted for insitu stabilisation works of this type when the parent material to be stabilised is a marginal sandstone material liable to break down under load.

There is a need to refine laboratory testing procedures in order that closer agreement can be achieved between the results of testing of laboratory-moulded specimens and field cores if protocols are to be developed which will allow field performance to be successfully predicted through laboratory testing.

In general, the performance of both the bitumen/cement and slag/lime materials was satisfactory, with no fatigue cracking or subgrade deformation observed. Interpretation of the results was, however often hampered by the early fatigue failure of the thin asphalt surface and also by local deformation and cracking at the interfaces of two test sections. For these reasons, it was not possible to estimate the life of either material.

5.15 Fatigue Properties of Asphalt Mixes

Between 1996 and 1998, Austroads and the Australian Asphalt Pavement Association (AAPA) sponsored a series of accelerated loading trials, using the Accelerated Loading Facility (ALF), to assess the fatigue performance of a range of asphalt mixes typically used in Australia.

The initial study involved a series of trials of asphalt mixes composed of a conventional Class 320 binder and a modified SBS binder. The SBS mix was selected as an example of a mix incorporating a polymer modified binder which would have a longer life than the conventional mix. The trial pavements constructed comprised a relatively thin 35 mm to 50 mm asphalt surfacing, a nominal 150 mm thick unbound crushed rock base layer, and a 500 mm thick sand sub-base layer. The test sites were constructed in such a way that ALF trafficking could be conducted simultaneously on both mixes. Following the completion of this initial trial, it was agreed that there was a need for further evaluation of the conventional Class 320 mix, and a stiffer (Class 600) mix under both field and laboratory conditions.

About 500,000 cycles of the ALF trafficking using the 80 kN dual-wheel load (equivalent to about 8 million standard axle repetitions) were applied simultaneously to the mixes during both the initial and follow-up trials. However there was no increase in maximum deflection or curvature during the testing and no suggestion that any fatigue failure was being induced in any mix. The only distress observed during both trials was surface deformation towards the end of trafficking. As this deformation was mainly confined to the sand sub-base layer rather than to the underlying subgrade, it was not possible to offer any comments regarding the applicability, or otherwise, of the Austroads subgrade strain equation used in pavement design in this situation.

In addition to the ALF testing programme, a complimentary laboratory testing programme was undertaken of the mixes involved. The significant finding was that the fatigue performance of the Class 600 mix was only marginally superior to the conventional Class 320 mix.

A follow-up response-to-load testing programme was also undertaken using 50 kN and 80 kN dual wheel loads and a 50 kN super single wheel load. This testing indicated reasonable agreement between the measured and modelled strains at the bottom of the asphalt layers. However as this testing was carried out at a later time where the temperature was much higher than for the ALF testing, some unrealistic temperature correction factors had to be applied which hampered the analysis.

In terms of the derivation of 'shift factors' between laboratory predicted and observed field performance, 'lower bound' values of 3.8 for the Class 320 binder and between 2.4 and 6.3 for the Class 600 binder were suggested. However it was not possible to derive actual shift factors because none of the four pavements tested by ALF during both stages of the trial failed due to fatigue of the asphalt surfacing layer.

Although the trial did not produce any definitive answers in terms of the applicability of the asphalt fatigue relationship contained in the Austroads Pavement Design Guide to current practice, the trial was successful in that it established that the relationship was conservative. Further work is required, particularly a literature review to determine if similar studies have been undertaken elsewhere, especially those involving the derivation of asphalt fatigue 'shift factors'.

5.16 Design Characteristics of Rigid Pavements

About $40M is spent annually in Australia on new construction of concrete pavements, largely on the basis that long life, low maintenance highways will result from this large capital investment. Whilst some studies are currently being conducted into the long-term performance of concrete pavements, most notably as part of the US Strategic Highway Research Programme, several issues had recently been identified that could be addressed if the Commonwealth and the States are to acquire a higher degree of confidence in the longer-term performance of concrete pavements in Australia.

In order to address these issues, a project titled, "Design Characteristics of Rigid Pavements" was commissioned by Austroads in 1997/98. The major objectives of the trial are to:

- validate and refine the current fatigue design criteria for plain concrete pavements (PCPs) (Part 1); and
- investigate the impact of the performance of a range of sub-base types on the performance of PCPs (Part 2).

As an adjunct to these objectives, two other issues are being addressed:

- the role that dowels play in load transfer, and the effects on slab curling; and
- the role of tied shoulders on the performance of PCPs.

Testing to date in Phase 1 of the Fatigue Trial has consisted of an investigation of the fatigue behaviour of four PCPs (on a lean mix concrete sub-base) using the ALF and a rigid truck, the latter being used, in association with ALF, for response to load testing.

To date, four experiments have been conducted using the ALF and no fatigue failure could be induced under the ALF 80 kN dual-wheel load. The response to load testing involved an investigation of the influence of design parameters (slab thickness and the presence of dowels and shoulders), load (magnitude, configuration and location) and climate (particularly temperature) on the behaviour of PCPs. Strain gauges were placed within, and on the surface of, the concrete slabs, and partial deflection gauges were installed to measure the deflection at the base/sub-base interface. Temperature data was collected at the top, middle and bottom of the slabs.

This testing has pointed to the influence of temperature on the behaviour of the slabs, the different behaviour under the (half) single-axle ALF compared to the (full) single-axle truck, the influence of the shading effect of the ALF on response and the need to properly calibrate the strain gauges in order that more meaningful results can be obtained.

In terms of the relative effects of dowels on thermal strains and differential thermal strains, it was found that both thermal strains and differential thermal strains along an undowelled transverse joint were more than three times the strains along a dowelled transverse joint. Partial deflection data collected at the interface of the base and sub-base of a slab 200 mm thick, with undowelled transverse joints, also indicated that movements at corners on the side without shoulders were up to 50% greater than those at corners on the side with shoulders.

The second phase of testing – during which the relative performance of the various sub-bases ('erosion trial') was examined – was completed in March 2000. Analysis is almost complete and the results will be available shortly.

5.17 Effect of Maintenance on Pavement Performance

Network level pavement life-cycle costing (PLCC) analyses show that the rate of pavement deterioration has the greatest impact on the annual agency costs of maintenance and rehabilitation. These analyses show that potential annual agency budget savings of up to 39% exist (A$795 million per year in Australia) if current average rates of pavement deterioration are reduced to the lowest observed rate (0.2 to 0.4 NAASRA Roughness/year). However, the cost of achieving these budget savings by increasing maintenance to reduce pavement deterioration is unknown and will remain so until the influence that maintenance has on pavement deterioration is quantified.

Quantification of the influence that maintenance has on pavement deterioration has not been possible because:

- existing pavement deterioration models that are used in network level PLCC analyses are poor predictors of future deterioration due to the correlation of the data used in developing these models;
- the data available from pavement management system (PMS) data bases are not suitable for developing deterioration models because this data is correlated; and
- deterioration models based on existing data reflect current maintenance strategies and practices, and therefore cannot predict the effects of marked changes in those practices.

The specific aim of this project is to undertake a series of pilot accelerated loading tests on existing pavements, using ALF, to examine the influence of specific maintenance treatments on pavement deterioration. Due to the accelerated loading, the influence of the environment will be largely excluded from this test. The pilot ALF testing programme was based on the recommendations developed to define the testing procedures and failure criteria under a specific range of maintenance treatments.

An initial trial was successfully completed in September 1998 and a second trial was conducted in April 1999. Pavement performance (roughness, rutting, cracking and deflection) was continuously monitored along with loading and environmental conditions. The aim of the testing was to confirm the feasibility of using ALF to quantitatively predict the impact of maintenance on pavement performance (roughness, rutting and strength) via links with LTPP sites across Australia. During ALF testing all test surfaces will be subject to light surface water simulating rainfall and a consistent environmental condition.

The latest series of trials commenced at the permanent site in Melbourne in June 2000. Testing is being conducted inside a shed that has now been constructed in order that the option for using ALF indoors is available.

The expected output of the project will be reliable pavement performance models for use at network and project levels to set maintenance and rehabilitation intervention strategies that reduce pavement deterioration and life-cycle costs.

6. INTERPRETATION OF FULL SCALE TESTING RESULTS

Pavement tests at ALF and CAPTIF are a complex form of experiment, often involving a large number of parameters whose importance is to some extent variable according to the particular purpose of the test; such tests provide valuable data for complete pavement performance studies. In this respect, it is important to ensure that data are collected in the most comprehensive and convenient form. The data collected fall into various categories such as materials, pavement structure, traffic loadings, climate and performance (structural and functional deterioration) and have been described extensively elsewhere in this report.

Because the performance of the pavement depends on how pavement characteristics such as layer thickness, moduli, etc. vary from one station in the test pavement to the next, the amount of variation and pattern of variation along the length of the test pavement should be measured in addition to the mean value, for many of the parameters.

6.1 Use of Test Results

Accelerated pavement testing may be used to improve empirical performance relationships, but ALF and CAPTIF projects usually differ from in-service conditions in several respects. When developing or verifying theoretical models for predicting pavement response these differences are of little importance, but for empirical relationships between response and performance, they are of crucial importance.

Accelerated pavement testing provides the link between laboratory testing on small samples and in-service pavements. The advantages of accelerated pavement testing compared with laboratory testing are that conditions of load, structure and other factors are more realistic, and the advantages with respect to observations of in-service pavements are that conditions of climate and loading may be closely controlled and accelerated.

The main differences between accelerated pavement testing and testing of in-service pavements are as follows:

- Traffic loading is accelerated, but a number of other changes cannot be accelerated, such as changes in moisture balance and ageing of bitumen.
- For CAPTIF, which has limited climate control, some climate conditions such as sun and rain are neglected. For ALF, which has little climate control, testing is limited to those climatic conditions that prevail during testing, and extreme climatic conditions cannot be reproduced (although the development of the pavement heating system has assisted in specific studies and, more recently, attempts have been made to chill pavements).
- Owing to cost restrictions, testing can be conducted only for a very limited number of combinations of pavement materials, subgrade materials, layer thicknesses, climatic conditions and loading conditions. In-service pavements experience a much wider spectrum of these variables than it is possible to impose in full-scale testing. Even the most extensive and expensive full-scale test ever carried out, the AASHO Road Test, covered only one subgrade and one climatic (freeze-thaw) condition.

7. COMPARISON OF FACILITIES

7.1 Field Measurements and Instrumentation

Table 7.1 compares the field measurements carried out during and after construction and during and after testing, whilst table 7.2 lists the instrumentation that has been used during the testing programmes. It needs to be noted that some of the instrumentation has been supplied by one accelerated loading facility for use by the other, e.g. Bison Coils for an ALF axle load equivalency trial and H-bar strain gauges and Partial Depth gauges for Element 1 of the DIVINE project, which was conducted at CAPTIF.

Referring to table 7.1, load cells are provided with the ALF but the load actually applied to the pavement during trafficking is not currently recorded because the supporting system is due for replacement with a more modern system.

Moisture movement is measured during some ALF trials using Time Domain Reflectometry (TDR) gauges, a system developed by CSIRO and adapted for the specific conditions pertaining to these trials by Queensland Department of Main Roads during the early 1990s. This system could easily be adopted to use at CAPTIF in a situation where moisture movement was a parameter.

In terms of table 7.2, both facilities use a specially-developed system for measuring transverse profile. ARRB Transport Research owns a Falling Weight Deflectometer (FWD) which was manufactured especially to fit under the ALF mainframe. This means that the ALF does not have to be moved off the test site before deflection testing can proceed. Transit NZ do not own a FWD but have access to a device which is owned locally in New Zealand. The deflection data is used to both assess the variability of the pavement in terms of its bearing capacity and in back-calculation studies to determine the modulus of the various pavement layers, both at construction and over time.

Bison coils have been used extensively with much success during CAPTIF trials for measuring vertical stress within the pavement layers. These coils were trialed during one ALF trial but, unfortunately, meaningful data could not be collected. This was probably related to the high moisture contents prevailing in the site used and the fact that ALF was not located indoors.

Multi-depth deflection gauges were used during early ALF trials but their use has been discontinued owing to the difficulties associated with installing them into the pavement, the maintenance of the access holes into which they are placed and the doubts generally as to the quality of data produced compared to the effort required to collect that data. Partial depth defection gauges – i.e. gauges that measure deflection of the surface with respect to a specific pavement layer – have, however, been successfully used in a number of ALF trials and were also used at CAPTIF during the DIVINE Project.

Cracking is collected manually on clear plastic sheets, with a different colour used to depict the development of cracking over time. Cracking is reported in terms of area of cracking per square metre of trafficked surface. Automatic methods of measuring cracking have not been developed for use at either site.

Table 7.1 Measurements during and after construction and during and after loading

			ALF	CAPTIF
1.	During and after construction			
	1.1	Surface geometry	Y	Y
	1.2	Density of layers	Y	Y
	1.3	Deflection	Y	Y
	1.4	Bearing capacity/CBR	Y	Y
2.	During loading			
	2.1	Load applications	Y	Y
	2.2	Load magnitude	N	Y
	2.2	Operating statistics	Y	Y
	2.3	Temperature		
		2.3.1 Surface	Y	Y
		2.3.2 Air	Y	Y
		2.3.3 Pavement	Y	Y
	2.4	Rainfall	Y	N/A
	2.5	Deflection/Bearing capacity	Y	Y
	2.6	Deformation/Rutting	Y	Y
	2.7	Soil pressure	N	Y
	2.8	Stress/Strain	Y	Y
	2.9	Cracking	Y	Y
	2.10	Moisture movement	Y	N
3.	After loading			
	3.1	Deflection/Bearing capacity	Y	Y
	3.2	Deformation/Rutting	Y	Y
	3.3	Surface geometry	Y	Y
	3.4	Cracking	Y	Y
	3.5	Density of layers	Y	Y

Table 7.2 Instrumentation used at both facilities

	ALF	CAPTIF
1. Deflection/Bearing capacity		
1.1 Falling Weight Deflectometer	Y	Y
1.2 Loadman	N	Y
1.2 Benkelman Beam (or derivative thereof)	Y	Y
1.3 Multi-depth/partial deflection gauges	Y	Y
2. Surface Deformation/Rutting/Profile		
2.1 Specialised transverse profile devise	Y	Y
2.2 Walking Profiler	Y	N
2.3 Dipstick Road Profiler	N	Y
2.4 Single laser profile devise	N	Y
3. Cracking		
3.1 Automated	N	N
3.2 Manual	Y	Y
4. Within-pavement stress and strain		
4.1 H-bar strain gauges	Y	Y
4.2 Bison coils / Emu Soil Strain System	Y	Y
4.3 Soil Pressure cells	N	Y
5. Within-pavement moisture movement		
5.1 TDR gauges	Y	N

The Loadman is a portable device, similar to the Clegg hammer, which is used to measure surface deflection. The data is recorded and an algorithm is used to calculate a typical modulus of the pavement at the point tested. The device is particularly useful for evaluating the variability of the pavement layers and, in this application, it is used in a similar manner to the FWD.

7.2 Laboratory Measurements

Table 7.3 shows the laboratory tests that are conducted in association with the field testing. The extent, and type, of testing obviously depends on the nature of the trial. Routine laboratory testing (compaction, Atterberg limits, etc.) is conducted during all trials.

Table 7.3 Laboratory tests conducted in association with trials

	ALF	CAPTIF
1. Subgrade		
1.1 Compaction	Y	Y
1.2 Soaked/unsoaked CBR	Y	Y
1.2 Atterberg limits	Y	Y
1.3 Triaxial testing	Y	Y
2. Unbound materials		
2.1 Compaction	Y	Y
2.2 Soaked/unsoaked CBR	Y	Y
2.2 Atterberg limits	Y	Y
2.3 Triaxial testing	Y	Y
3. Cemented materials		
3.1 Elastic modulus (indirect tensile & fatigue)	Y	Y
3.2 UCS	Y	Y
3.3 Beam testing for fatigue life	Y	N
4. Asphalt		
4.1 Elastic modulus (indirect tensile & fatigue)	Y	Y
4.2 Uniaxial creep	Y	Y
4.3 Beam testing for fatigue life	Y	Y
4.4 Bitumen content/air voids	Y	Y
4.5 Binder characterisation	Y	Y

Over recent years, a great deal of research and development work has been conducted in the area of materials characterisation and this has led to the development of standard test equipment (based on the Materials Testing Apparatus (MATTA) – and also the UMATTA) and standard tests for the determination of the principal properties of the materials (elastic modulus, fatigue life, etc.). Since the early 1990s, almost all trials have included a comprehensive materials characterisation programme. In fact, some trials have been specifically tailored to address laboratory-based issues; for example, the ALF trial of the deformation properties of asphalt mixes was designed to assess the suitability of the dynamic creep test as an asphalt mix design tool.

7.3 Topics Addressed in Trials Conducted to Date

The Austroads Pavement Reference Group (APRG) is responsible for the development, management and updating of the *Strategy for Pavement Research and Development* which determines research and development activities in the areas of pavement and materials performance. This work is funded by Austroads – through its Technology and Environment Programme – and supported by the asphalt (through the Australian Asphalt Pavement Association), cement (through the Cement & Concrete Association of Australia) and stabilisation (through the Australian Stabilisation Industry Association) industries. This work is conducted in cooperation with Transit New Zealand, which is a member of Austroads, and the New Zealand Pavements and Bitumen Contractors Association, a member of APRG.

Table 7.4 lists the projects conducted to date in terms of the research topics addressed in the Austroads Strategy. Also shown are trials which addressed 'asset management' issues such as life cycle costing and maintenance intervention practices. Such work in Australia is generally funded through the Austroads Business Systems Programme.

In all cases, the major topic addressed in the trial is listed, realising that some trials also addressed other issues (e.g. trial A3 led to changes in QDMR construction practice for cemented layers and one of the outputs of trials A14 and A15 was draft design charts for deep lift recycled and stabilised flyash pavements respectively.

It can be seen from table 7.4 that the topics addressed to date by the two facilities – and planned for the immediate future – reflect:

- the primary issues of concern to Australasia that can be addressed using accelerated pavement testing (traffic loading, unbound materials, marginal materials, rehabilitation, etc.);
- the capabilities of the two facilities (CAPTIF emphasis on 'loading' issues; ALF emphasis on 'rehabilitation' issues); and
- the increasing importance of 'asset management' issues and the need to generate 'Australasian' data for input into existing pavement performance models.

Although no trials have specifically addressed 'pavement performance' as the major issue, all trials, of course, rely on the performance of the pavement being monitored in order that the issues can be addressed. The title of the topic is also misleading, to the extent that the emphasis in the Austroads Strategy is placed on the performance of in-service pavements under 'real' traffic in order that aspects that cannot be easily addressed with accelerated loading (viz. the seasonal effects of climate and traffic on performance) can be addressed. In addition, 19 in-service test pavements, including several pavements previously monitored using ALF, are currently being monitored in Australia as part of the Austroads National Strategic Research Programme. Many other in-service pavements are also being monitored by local agencies within Australasia.

In examining the above data, one issue that has not been seriously addressed by either facility is 'Surfacings', with the ALF work to date restricted to studies of geotextile reinforced seals and granite sett pavements. The use of thin surfacings as a rehabilitation treatment is one issue well suited to accelerated pavement testing.

Table 7.4 Topics Addressed in Trials Conducted to Date

Austroads Topic	ALF Trial	ALF Title	CAPTIF Trial	CAPTIF Title
Traffic loading	A10 A16	crushed rock axle load equivalency	C2	Comparative rutting: duals and wide-base single tyres
			C8/C10	Dynamic wheel loads and pavement wear
			C9	DIVINE (Element 1): air bag and multi-leaf suspensions
			C13	Comparison of 8 t and 10 t axle loads
Road making materials (asphalt and binders)	A5	Asphalt rehabilitation treatments	C6	Modified binders in asphalt mixes
	A13	Rut-resistant properties of asphalt		
Road making materials (unbound & recycled materials)	A1	heavy duty unbound pavements with thin asphalt surface	C1	Unbound granular pavements under chip seals
	A2	heavy duty unbound pavements with chip seal surface	C3	Effects of particle shape and gradation on basecourse performance
	A4	Unbound and stabilised blast furnace slag	C3	Cross Test
	A8	Fine grained marginal materials		
	A12	Lateritic gravel bases and sub-bases		
	A15	Stabilised flyash pavements		
Road making materials (stabilised materials)	A3	Cement-treated bases and sub-bases	C4	Lime-stabilised sub-bases
	A9	Bitumen and bitumen/cement-stabilised crushed rock		
	A17	Insitu stabilisation of marginal sandstone		
Pavement design & structural behaviour	A6	Fatigue properties of asphalt and cement-treated crushed rock	C5	Strain response of subgrades and unbound granular pavements: wheel load, tyre pressure, tyre type
	A18	Fatigue performance of asphalt		
	A21/A23	Design characteristics of rigid pavements		
Road surfacings & markings	A7	Geotextile reinforced seals		
	A20	Granite sett pavements		
Construction/maintenance including rehabilitation	A11	Crushed rock moisture/compaction		
	A14	Deep lift insitu recycling		
Asset management (Business Systems program)	A22/24	Maintenance intervention strategies		

7.4 Capabilities of Facilities

In terms of the capabilities of the two facilities – in other words, their suitability for addressing issues of importance to Australasia – table 7.5 compares the capabilities of the two systems.

The differences in the two facilities obviously mainly relate to the type of pavements that can be tested, and the type of loading and the environmental conditions under which the pavements are tested.

CAPTIF is clearly ideal for 'ranking' studies because up to three pavements can be tested at the same time and, if necessary, two magnitudes of loading can be applied via two suspension types at a range of speeds over two different wheelpaths. These capabilities also make it ideal for 'loading' studies such as axle load equivalency, tyre type, tyre pressure, vehicle dynamics, etc. Axle load equivalency trials have been attempted using ALF with only limited success owing to the problems associated with the control of moisture. In terms of material type, the CAPTIF facility is probably most suitable for unbound materials and for studies of asphalt performance not requiring testing at high temperatures (e.g. deformation studies).

ALF, on the other hand, is probably more suited to 'construction/rehabilitation' type of issues, because pavements are constructed using normal construction plant and procedures, and processes such as insitu stabilisation can be conducted far more easily on these types of ALF sites rather than the permanent facility at CAPTIF. The number of ALF trials that have addressed the suitability of marginal, recycled and stabilised materials reflects this. ALF is also obviously more suited to trials involving the introduction of external parameters such as the dams used in the evaluation of geotextile reinforced seals at Brewarrina, NSW (Johnson-Clarke, Sharp and Walter 1993).

Possible fields of study which could be considered and which probably require particular features of the facilities are as follows:

- Rutting of asphalt and high temperature effects – perhaps more suited to ALF because of its pavement heating system.
- Changes in soil and subgrade conditions – more suited to CAPTIF because of its 'deep pit' facilities.
- Variations in pavement structures – in principal suited to both facilities. In the case of treated basecourse materials a sufficient section length is required for the observation of discontinuities or the possibility of load application at the discontinuity with a mobile facility.
- The effects of varying loads and the accumulation of damage – in principal both facilities but more suited to CAPTIF because of its control of moisture environment.
- Road condition rating instruments – in principal suited to both facilities.
- The use of non-conforming materials and the revision of performance-based specifications – possibly more suited to ALF for field trials and CAPTIF for comparative trials.
- The use of thin surfacings as a pavement rehabilitation treatment – suited to both facilities.

Table 7.5 Comparison of capabilities of facilities

ALF	CAPTIF
Loading	
Half-single axle load	Half-single axle load
Modified air bag suspension only	Air bag and steel spring suspensions providing realistic dynamic loading which is recorded
Provision of a tandem axle is not a viable option	Commissioning of half-tandem axle load an option in original design but not introduced to date
Load can be varied between 40 and 80 kN in 10 kN increments	Load can be varied between 21 and 60 kN in 3.3 kN increments
Loading can be applied with either dual tyres or wide single tyres	Loading can be applied with either dual tyres or wide single tyres
Covers range of loading likely to be encountered in service – higher loads useful for undertaking certain studies, particularly those involving the relevant 'ranking' of a range of materials or treatments	Covers range of loading likely to be encountered in service – higher loads useful for undertaking certain studies, particularly those involving the relevant 'ranking' of a range of materials or treatments
Loading is applied linearly in one direction	Loading is applied in a circle
Loading is applied over a 12 m length of pavement at a speed of 20 km/h – this equates to about 380 cycles/hour or about 45,000-50,000 cycles per week	Loading is applied by two SLAVES over an average circumference of about 58 m at speeds up to 50 km/h – this equates to 775 cycles/hour (at a speed of 45 km/h) or about 100,000 cycles per week
Response to load testing can be conducted at creep speed	Response to load testing can be conducted at creep speed
Loading can either be channelised or distributed transversely over a pre-set pattern	Loading can either be channelised or distributed transversely over a pre-set pattern
Loading confined to one wheelpath at any one time	Loading from each SLAVE can be applied over different wheelpaths which allows effects of different suspensions, etc. to be evaluated at the same time
Site Conditions	
Testing can be conducted either on existing in-service pavements or on test pavements specially constructed at 'off-road' sites	All testing is conducted at a permanent site in Christchurch
Test pavements are constructed full-scale in a linear direction using normal construction plant	Test pavements are constructed inside a circular concrete pit and generally mid-sized construction plant is used
Construction standards achieved (compaction, etc.) are in line with normal standards expected in the field	Construction standards achieved (compaction, etc.) are in line with normal standards expected in the field
Until June 2000, all testing was conducted outdoors – control of environment, particularly moisture, was a problem (shed commissioned in June 2000)	All testing is conducted indoors – environment, particularly moisture, is therefore controlled
Pavements can be heated to a temperature of 60±2°C	Heating of pavements not attempted to date
Some success with controlled cooling of pavements but application limited and costly to operate	Controlled cooling of pavements not attempted to date
Up to three different pavements can be tested at the same time but success strongly dependent on stiffness of base layers and length of pavements tested (4 m) small – problems can occur (e.g. variable roughness/dynamic loading) if pavements fail	Up to four different pavements can be tested at the same time and length of each pavement tested (up to 15 m) more than adequate in most situations – problems can occur (e.g. variable roughness/dynamic loading) if pavements fail. If a pavement section fails, the dynamic loading is recorded to determine if there is an effect on the adjacent sections
Multiple sections can be constructed for future testing and testing can continue whilst new sites are being constructed	Test sites confined to what have been constructed in the pit – subgrade and lower layers can be re-used in later trials

- Comparative studies of maintenance on the same pavement structure where differences in level of the tested surface are permitted – best suited to CAPTIF as transition ramps could be built between different height sections of pavement. ALF would have to conduct a series of tests to complete this type of testing.
- Load effects after structural maintenance works – currently being examined by ALF in a pilot study.
- Studies on serviceability of pavements – this would require long test sections to eliminate/minimise the variability of construction – probably best suited to CAPTIF due to the longer test section that can be tested at one time.

8. Details ALF/CAPTIF Cross-Test

8.1 Details of ALF Benchmark Test

A recently completed ALF trial was selected to provide a benchmark to enable the comparison of performance of the two facilities. The 16th and 17th ALF trials were conducted at a test site located at the VicRoads Fowler Rd. Depot, Dandenong, about 30 km south-east of Melbourne. A full description of the trials is provided in Vuong and Sharp (1997). The 17th trial was deemed to be the benchmark trial. Some of the main objectives of the trial were to:

- determine the deformation 'life' of all pavements and the fatigue 'life' of the bound pavements, and to compare these findings with the recommendations currently contained in the Austroads Pavement Design Guide (Austroads 1992);
- develop indicative performance characteristics in the laboratory with a view to enabling the selection of the most appropriate binder type and proportion on a mechanistic basis;
- compare the performance of the unbound materials characterised in the laboratory with the ALF trial results and the predicted lives based on the Austroads Pavement Design Guide procedures;
- if possible, determine axle load equivalency factors and derive relative damaging factors when loading is applied via the ALF dual-wheel and single-wheel loads.

8.1.1 Layout of Test Pavements

Table 8.1 details the pavement structure used during the 16th trial. Pavement S1-1 was used exclusively for the load equivalency trials.

Table 8.1 Details of test pavement

Pavement ID	Pavement Description	Purpose
S1-1	20 mm non-plastic crushed rock	axle load equivalency trial - dual-wheel loading

The test pavements were constructed on an area previously used by VicRoads road maintenance staff for the storage of pavement materials and similar facilities. It consisted of a variety of materials laid in an ad hoc fashion over a number of years. Whilst many areas of the existing pavement were very strong (maximum deflections 0.1-0.5 mm), it was also very variable (asphalt thickness 40-100 mm; thickness to subgrade varying from 300-600 mm). In other unpaved areas, the thickness of pavement (contaminated crushed rock) was about 150-250 mm and DCP CBR values varied between 7 and 50.

Accordingly, the existing pavement was removed down to the existing subgrade level. Because this material was a low-strength silty material, it was stabilised with 2% lime to a depth of 300 mm prior to the installation of a free-draining layer and the imported subgrade in order that uniform support to the trial pavements could be provided (see section 8.1.2).

8.1.2 Cross-section of the Test Pavement

Figure 8.1 shows the cross-section of the test pavement. Referring to figure 8.1 the pavement consisted of:

- an asphalt surfacing (nominally 30 mm thick),
- a base of crushed rock or marginal material (nominally 200 mm thick),
- an imported clay (nominally 400 mm thick),
- a free-draining, permeable layer (nominally 100 mm thick),
- the natural clay subgrade stabilised to a nominal depth of 300 mm, and
- the natural subgrade.

The laboratory properties of the base materials are discussed in table 8.2. The permeable layer consisted of a 10 mm single-sized aggregate. A geotextile separation layer was provided both under and over this layer. The free-draining layer was provided to allow the subgrade to be wetted-up at a later date if necessary. To this end, provision was also made during construction to allow only part of the site to be wet-up if required.

A more detailed report on the construction of the test sites is contained in Sharp *et al.* (1997).

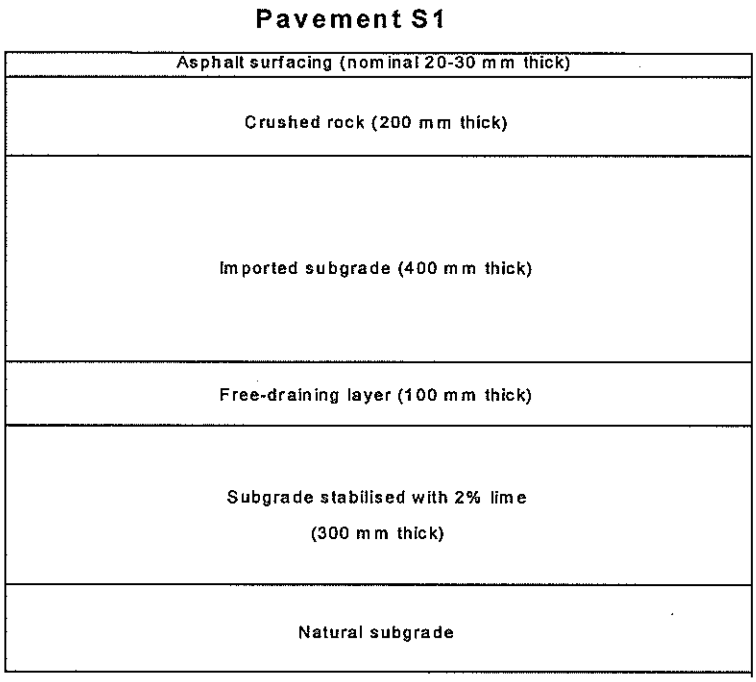

Figure 8.1 Cross-section of the test pavements S1-1 and S1-2

The imported subgrade was a Silurian clay, typically found in the Melbourne area, of nominal CBR 8-10 (dry) and a soaked CBR of about 3. The results of classification tests and Standard compaction tests conducted on the subgrade are also given in table 8.2.

Table 8.2 Laboratory properties of base and subgrade materials

Material	Plasticity Index (PI)	Liquid Limit (LL)	Linear shrinkage (LS)	Maximum Dry Density (t/m^3)	Optimum Moisture Content (%)	Unsoaked CBR	Soaked CBR
Crush Rock Base	0	21-23	1-3	2.2*	6.8*	118	166
Imported Subgrade	4-5	16	1-3	2.08**	8.6**	10	3

* Modified compaction. ** Standard compaction.

In the experiment, the narrow transverse ALF load distribution was used. The ALF load distribution follows a normal distribution resulting in a test width between the outer edges of the tyres of 900 mm.

During the ALF experiment, surface deflection bowls and surface profiles were measured at regular intervals using the Falling Weight Deflectometer (FWD) and the ARRB TR transverse profilometer respectively. Benkelman beam deflection testing was also attempted at the commencement of the trial but this testing proved to be too dangerous with the ALF loading because of the steep (1.5%) grade. Where cracking occurred during an experiment, crack maps were made on clear plastic sheets and the length of cracking recorded and reported in terms of crack length per square metre of pavement (m/m^2).

A number of Partial Deflection Gauges (PDG), Bison Coils, and pressure cells were installed within the pavement layers. For various reasons, the amount, and quality, of the data collected from the trials were very limited (Vertessy 1997) and varied significantly for the same pavement condition. Because the causes for such variation in the test data could not be identified, they were not used in the analysis and are not discussed further in this report.

Following the completion of the experiments the test strips were excavated to determine the mode of failure and to assist in the determination of the reasons for the failures.

All the data collected during testing is tabulated in Vertessy (1997). Details of the experiment follow.

8.1.3 Result of Experiment 1702

In this experiment, the 40 kN dual-wheel load was used. Heaving was observed at the two edges of the ALF loading area throughout the loading history (before and after cracking occurred), indicating that shear failure was occurring (see figure 8.2). The deformation and heave near the pavement edge were much higher than that on the other side of the wheelpath, probably due to higher moisture contents near the pavement shoulder, as discussed shortly, and also to the lack of lateral support.

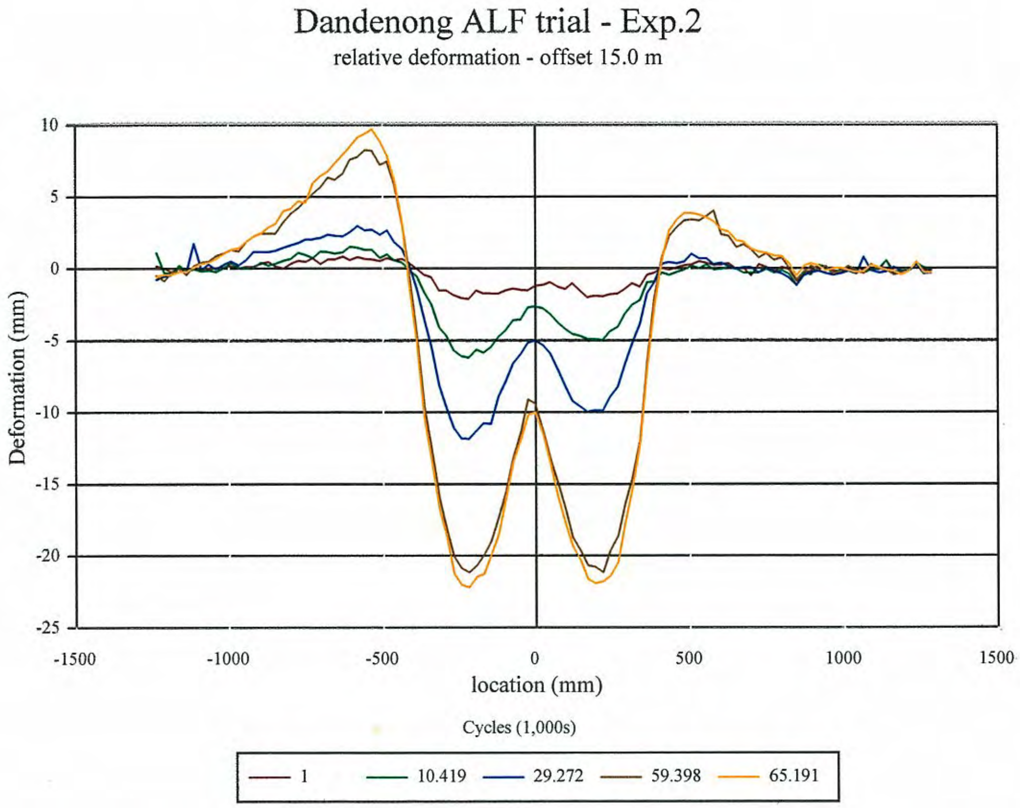

Figure 8.2 Variation in mean maximum surface with cycles of load (Experiment 1702)

Extensive surface cracking appeared after only 20,000 cycles, followed by an increase in the rate of deformation and shoving at the edges of the test section. The appearance of this cracking was a cause for concern because, at the prevailing low ambient temperatures, the predominant failure mode was fatigue of the thin asphalt surfacing rather than the expected deformation of the crushed rock base or subgrade. The pavement heating system used in the asphalt deformation trials was therefore used after 44,000 cycles, and in subsequent experiments, and the temperature in the asphalt layer was maintained at 25°C in an attempt to prevent any fatigue cracking in the asphalt. A total of 65,000 cycles was applied, at which stage extensive deformation (17 to 35 mm) and cracking had been induced in the pavement.

After the conclusion of experiment 1702, experiments 1703 and 1704 were conducted on untrafficked areas of pavement S1-1. For experiment 1703, ALF was moved a further 6 m to the west and the load was increased to 50 kN. The experiment was terminated after only 6,434 cycles because of extensive surface deformation (15 to 26 mm) which rendered the site untraffickable. For experiment 1704 ALF was moved another 6 m to the west and the load was increased to 60 kN. This experiment had to be terminated after only 1,700 cycles due to extensive surface deformation (15 to 22 mm) again rendering the site untraffickable.

8.1.4 Post-Mortem

Table 8.3 summarises the development of cracking in Sections S1-1. In all cases, cracking was first observed on the surface well before the pre-determined termination point of the test, viz. an average maximum deformation of 20 mm. This indicates that, despite the use of the pavement heating system to maintain an asphalt temperature at about 25°C, the initial mode of failure in the sections was fatigue of the thin asphalt surfacing rather then the expected permanent deformation of the base layer and/or subgrade.

Table 8.3 Crack development in crushed rock Section S1-1

Experiment	ALF Dual-Wheel Load (kN)	Loading Cycle (kcycle)	Crack Length (m/m^2)	Maximum Deformation (mm)
1702	40	20	>1	>10
1703	50	6	>1	>15
1704	60	1.7	>1	>15

Figure 8.3 shows the relative deformations of the asphaltic concrete (AC) basecourse/crushed rock (CR) and subgrade at the end of the ALF experiment for Sections S1-1. These results indicate that a large percentage of the total pavement surface deformation (say at least 70% of the total surface deformation) occurred in the crushed rock layer rather then in the subgrade. In other words, subgrade deformation may not always be the most critical factor influencing the life of granular pavements. This observation is similar to that observed in previous ALF experiments (e.g. Vuong and Sharp 1997).

The field density and moisture content of the crushed rock base in Section S1-1 was measured with the nuclear density gauge in back-scatter mode after the placement of the crushed rock layer and after each experiments, and the results are given in figures 8.4 and 8.5 respectively. It can be seen from figure 8.4 that, after placement, the crushed rock was quite uniform and had a field relative density and relative moisture content of 100% and 50% respectively. The section met the target compaction specification of 98% of MDD_{mod}. It can be seen from figure 8.5 however, that the final moisture contents were much higher than the moisture contents after placement (up to 80% of OMC_{mod}).

Investigation of the causes for the increase in moisture content in the crushed rock indicated that:

- the base layers in Section S1-1 wet up after heavy rainfall which occurred prior to the sites being sealed; and
- the base layer in Section S1-1 wet up further due to water entry from the shoulder during heavy rainfall which occurred after sealing.

The density and moisture content of the subgrade after placement was measured with the nuclear density gauge and results from Section S1-1 are given in figures 8.6 and 8.7 respectively. It can be seen that, after placement, the subgrade had field relative densities and relative moisture contents in the range 95-105% of MDD_{std} and 100-115% of OMC_{std} respectively.

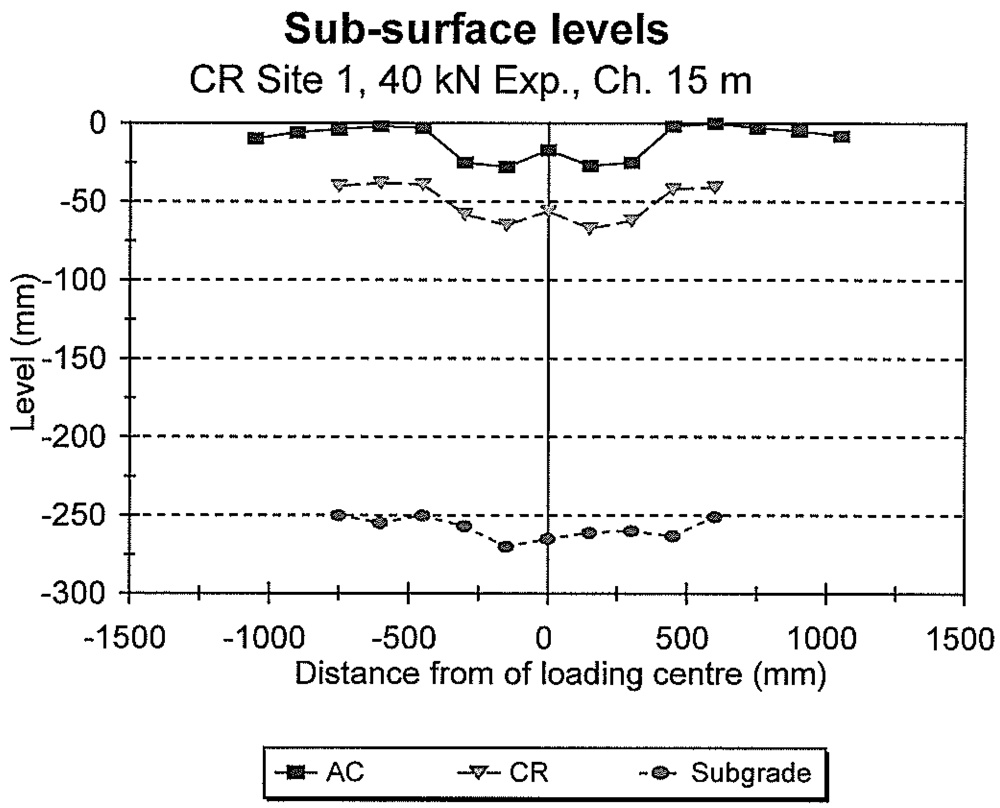

Figure 8.3 Sub-surface deformations in crushed rock Section S1-1 (Experiment 1702)

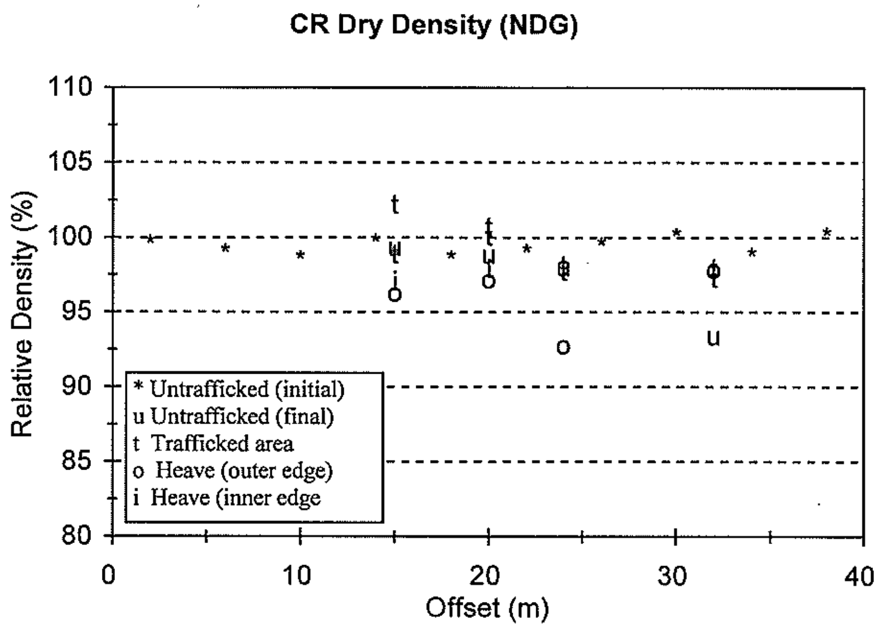

Figure 8.4 Variation in relative field density (% of MDD_{mod}) of crushed rock base in Section S1-1

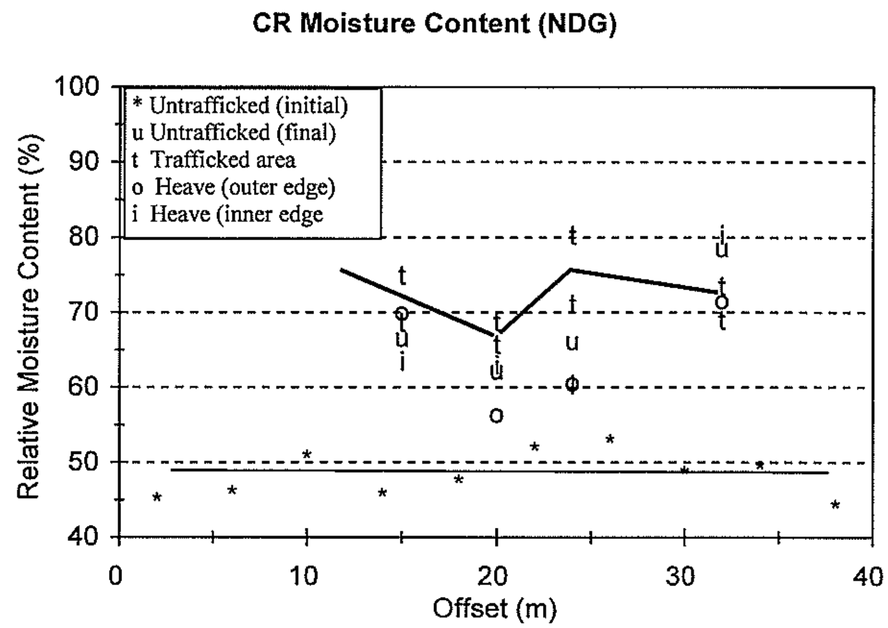

Figure 8.5 **Variation in relative field moisture content (% of OMC_{mod}) of crushed rock base in Section S1-1**

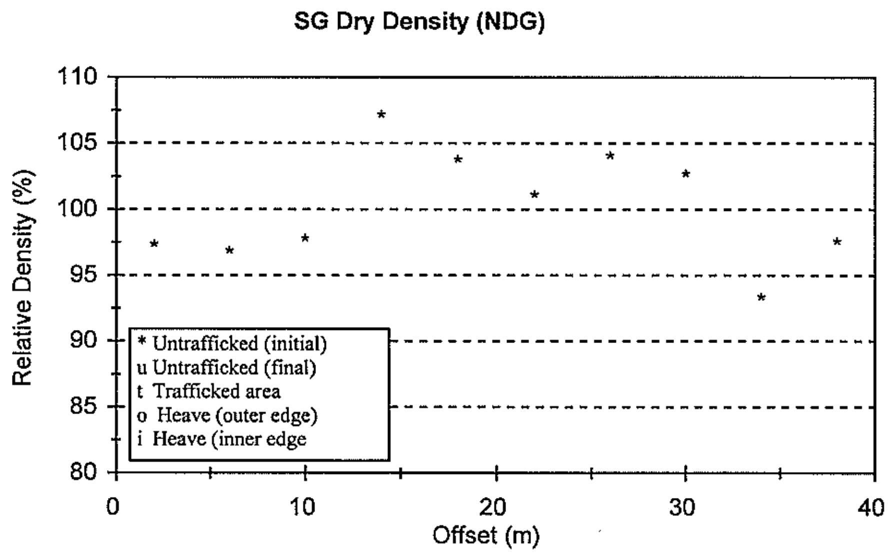

Figure 8.6 **Variation in initial relative density (% of MDD_{std}) of subgrade dry density in Section S1-1**

However, the final relative moisture contents of the subgrade in Section S1-1 after the completion of ALF testing were greater than 115% of OMC_{std}. As already discussed, an investigation of the causes of this increase in moisture content indicated that the subgrade layer had wet up after heavy rainfall before the site was sealed and also from the shoulder adjacent to Section S1-1.

The subgrade had CBR values in the 2-4% range, indicating that the subgrade was fully soaked.

Pavement deflections were measured with the FWD just prior to loading. The average initial value was 2.00 mm and the variation was within the limits of ± 0.10 mm. This indicated that the strength of the pavement was uniform at the commencement of loading.

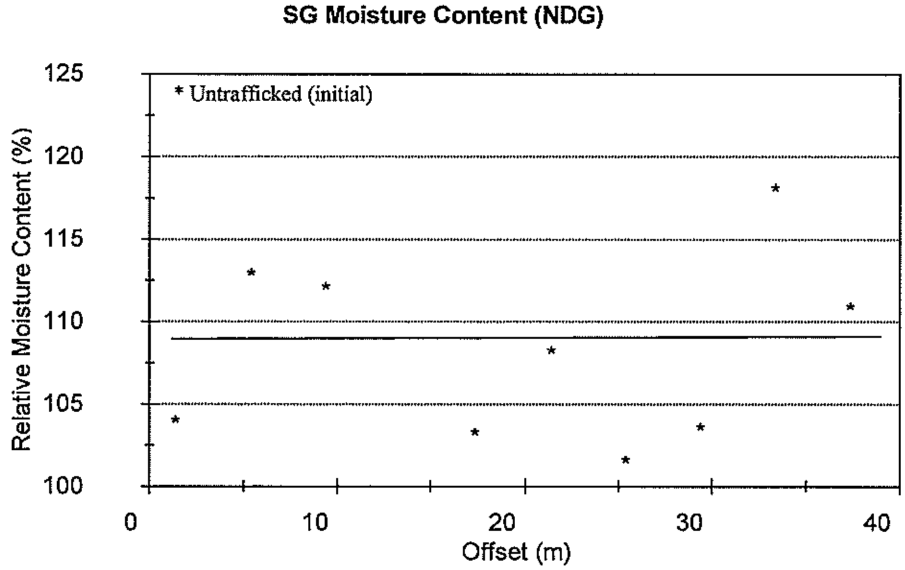

Figure 8.7 **Variation in initial relative density (% of OMC_{std}) of subgrade moisture content in Section S1-1**

In summary, the unbound granular pavement sections were constructed to the target thicknesses and dry densities. However, different sections had different moisture conditions due to water penetrating into the base layers and the subgrade through the surface prior to the sites being sealed and, particularly in the case of Section S1-1, from the pavement shoulder, which was adjacent to this section. The effects of moisture on the performance of unbound granular pavements are well known and very significant and, as a result, it was very difficult to compare the results obtained from the test sections because they had different pavement and subgrade moisture conditions. This once again illustrates the needs for a more controlled moisture environment during ALF testing. It also highlights the need to prime and seal unbound bases as soon as possible after they have been placed.

8.2 Cross Test

A test at CAPTIF was designed to duplicate a completed ALF test in order to investigate the effects or influences of the two different types of machine, and loading methods, on performance.

8.2.1 Test configuration

8.2.1.1 *Pavement design*

In order to minimise the amount of material that needed to be imported from Australia, it was decided to construct three different pavements of equal length in the tank. One section was constructed with the Australian material used in the benchmark test, another with material satisfying Transit New Zealand's AP20 M/4 basecourse requirements, whilst the remaining section was constructed with a combination of TNZ AP20 M/4 and an AP40 crushed rock.

Whilst the benchmark test described in section 8.1 involved the testing of a 200 mm thick layer of crushed rock, recent past experience at CAPTIF suggested that a 250 mm thick layer would be required in order to minimise the risk of early failure of the pavement. The combination section consisted of the AP40 sub-base and the TNZ AP20 M/4 basecourse. As the subgrade had progressively dried out over the previous four projects, it was decided to increase the moisture content of the material in an attempt to reduce the strength the material to a CBR value of 10%, consistent with the natural CBR value of the subgrade material used in the benchmark test in Australia. The pavement designs are shown in figure 8.8, whilst the layout of the three sections is shown in figure 8.9. This figure also shows the locations of the primary and secondary test sites.

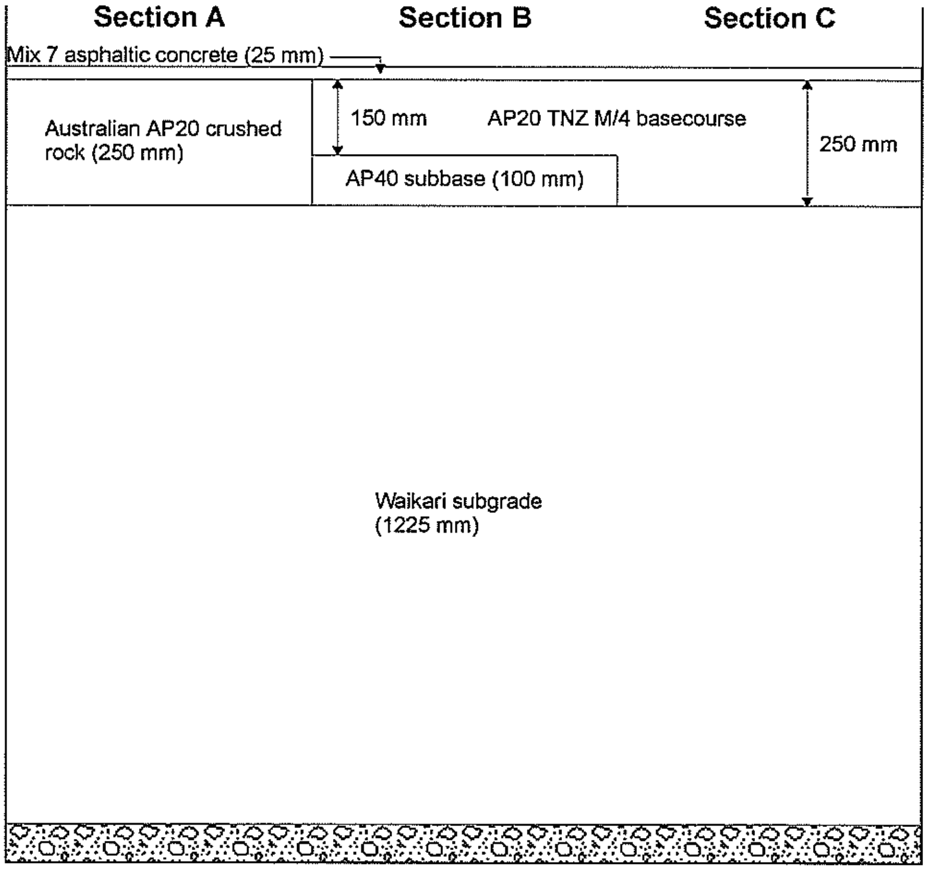

Figure 8.8 Pavement design for cross-test

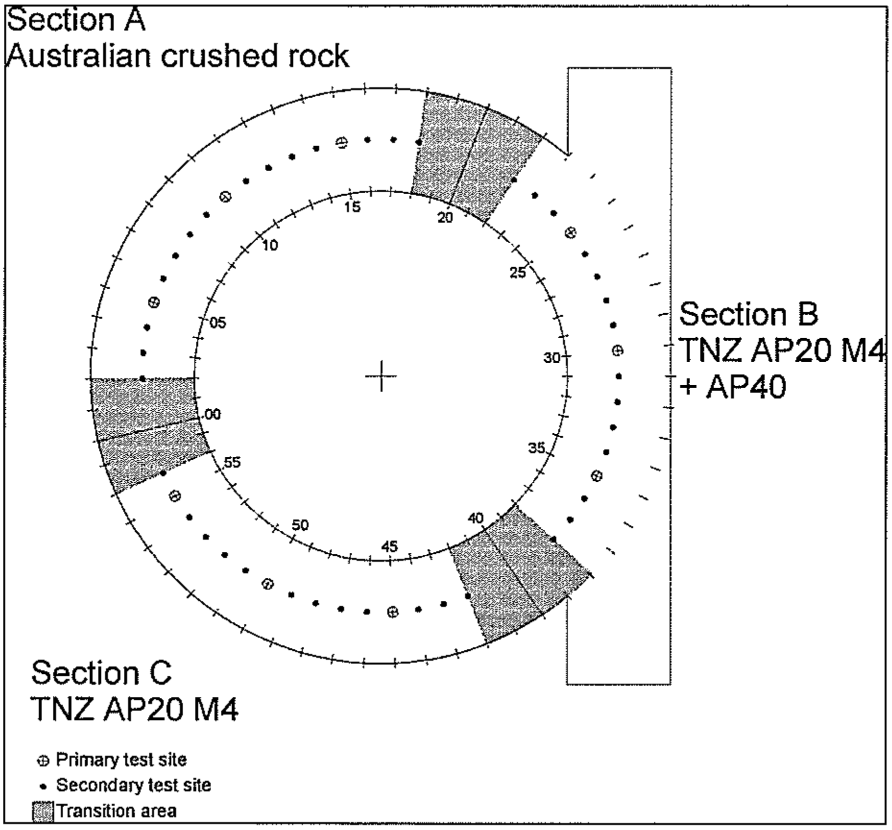

Figure 8.9 Layout of test sites

The primary test sites were to be monitored extensively during the project while the secondary test sites were only to be monitored at specific points. Section B was primarily a null section because, since the facility was upgraded in 1996/97, the provision of ramps into the pavement tank has resulted in the tangential section of the track at these locations being stronger due to the additional passes of the construction equipment over this section of track.

8.2.1.2 *Instrumentation*

Pavement surface profiles were measured in the transverse and longitudinal directions with the Transverse Profilometer and the Single Laser Profilometer respectively. Pavement surface deflections were measured with the CAPTIF Deflectometer at the primary and secondary test locations. Dynamic wheel forces were measured using the on-board accelerometer-based system. Vertical compressive strains in the pavement structure were measured with the Saskatchewan-developed Bison soil strain system. The strain coils were placed in the top 400 mm of the subgrade material in the sections containing the Australian and TNZ AP20 M/4 materials. An elevation view of the instrumentation is shown in figure 8.10.

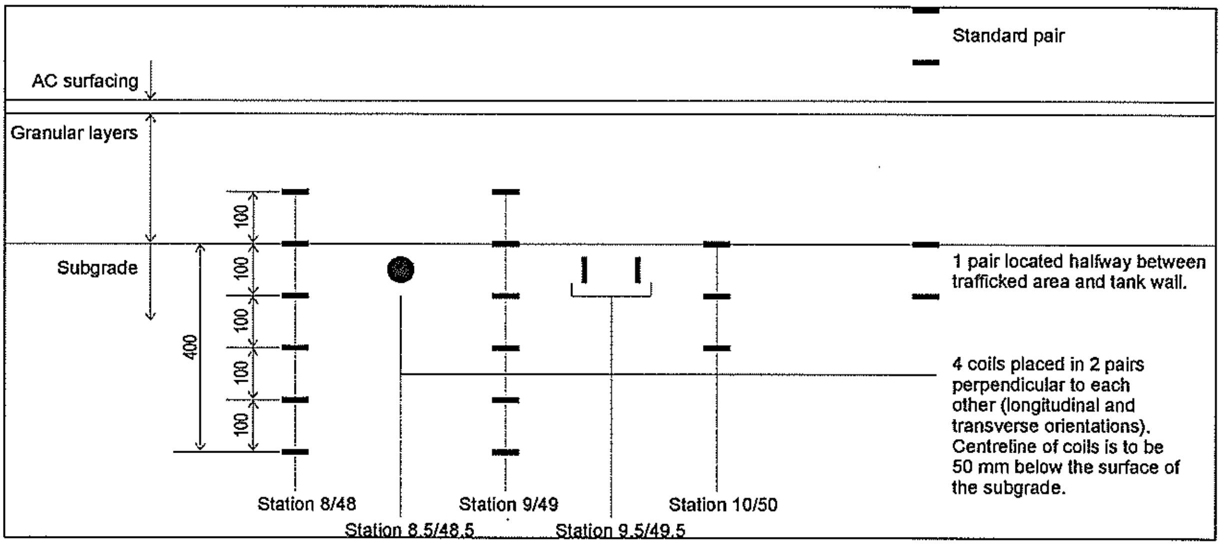

Figure 8.10 Layout of the Bison Soil Strain instrumentation (Sections A & C)

8.2.2 Pavement Construction

8.2.2.1 *Subgrade*

The previous test pavement was excavated from the tank, leaving approximately 500 mm of undisturbed Waikari clay in the bottom of the tank. The remaining material was ripped to a depth of 150 mm by a backhoe equipped with a ripper tooth. The moisture content was determined for the loose material and was found to be, on average, 7%. Water was added to raise the moisture content to 14%. A bulldozer was used to level the surface, after which it was rolled with a Ramex trench roller. Spot levels and Loadman readings were taken at primary sites. An external testing laboratory took backscatter nuclear density measurements at the primary sites. This lift was labelled Lift 1.

The clay for Lift 2 had been stockpiled on the concrete pad outside the test track. Sufficient material for lift 2 was spread in a thin (100-150 mm) layer and sufficient water was sprayed over the material to raise the moisture content closer to the OMC value of 14%. The material was placed in a 200 mm thick lift using the standard CAPTIF procedures as follows:

1. A pad was constructed at required design level for the bulldozer to sit on.
2. The material was brought in from the stockpile by a loader travelling over the previously-compacted surface.
3. The material in the loader bucket was dropped on the pad in front of the bulldozer then pushed over the edge of the pad by the bulldozer.
4. Heights were monitored by the laser level and digital staff throughout the backfilling stage, with an allowance of an additional 10% for compaction.

The moisture content was checked at a number of stations and additional water was added to bring the moisture content up to the desired value. The new sprinkler frame broke while in use and flooded the clay around Station 18. The remaining quantity of water was added using a hand-held sprinkler head. Due to time constraints, the sprinkler frame was not repaired and consequently not used for the rest of the construction programme. The wet material was removed and replaced with fresh material from the stockpile. The material was rotary hoed with a tractor-mounted hoe. The purpose of the hoeing was to break up the larger lumps and to thoroughly mix the water into the material. The material was levelled before compaction. Compaction of the lift was undertaken with the Ramex roller. The densities reached a plateau after two passes of the roller. Loadman tests and laser spot heights were measured on the primary sites.

The remaining lifts of subgrade material – lifts 3, 4 and 5 – were placed, each with a nominal lift thickness of 170 mm. During this period the weather was very hot and water had to be added to the material for each lift. Due to the time required to perform the moisture content testing, in conjunction with the hot weather, it was easy to add too much water and to increase the moisture content above the target value. This occurred on Lift 3 and, as a result, the Loadman results were slightly lower than the previous lifts. However the density measurements were high, even in the weaker areas (as indicated by the Loadman) areas. The moisture content from the NDM testing tended to be about 1.5% higher than the value obtained by drying with the microwave oven (no detailed record were kept, i.e. times, sample locations, etc.). The number of roller passes was increased to three on lifts 3, 4 and 5. The true compactive effort was probably higher as the Ramex roller was a skid steer roller. It was necessary to reverse the roller in order to change the direction and this probably increased the applied compactive effort.

At the completion of lift 4, Bison strain coils were placed in Section A, at stations 08, 09, and 10 and Section C, at stations 48, 49, and 50. Because each lift was being rotary hoed in-place, the coils had to be placed after compaction of the lift, a technique that had not been used before for Bison coils. Initially, an hand auger was used to auger down past the coil level and then a thin plaster base was poured to bring the pad for the coil back up to the correct level. This method, while giving good results, was far too slow. In addition, the effect of the larger plaster pad on the response of the coil was unknown. An alternative method was developed which involved using a hand auger to excavate a hole to the approximate depth, then levelling the base with a Proctor hammer, adjusting the level by adding more material as necessary. The use of the Proctor hammer allowed the prepared pad to be level, as the hammer could be held vertical during its use. During the placement of the coils, particular attention was paid to the placement of the cable in order to minimise the strain on the cable connection. Several methods were used to excavate a narrow trench for the cable, including a chainsaw (the chain wore out after one trench), an electric demolition hammer with a wide pick, and an electric circular saw with masonry blades fitted. The electric saw proved to be the most successful method, but it was very dusty as the masonry blade wore very quickly.

Lift 5, the final lift of clay, was placed in very hot conditions. This resulted in some uneven drying, especially near the doorways. Section C, near the sliding door, dried out too much, while the extra water that was added to cope with the dry conditions left the shaded Section A rather moist. Adding water to the surface of Section C only made it greasy and difficult to roll. Once the surface had been compacted, a steel frame was dragged over the entire subgrade surface to plane off any high spots. This made the surface more even but laser levels showed that some areas were too far below design level. The areas were backfilled with the Bobcat loader and hand raked. This method brought the levels up to the target values. However, when the fresh material was rolled, it would not knit with the clay below and the roller kept breaking the surface up when skidding to change direction.

The next day an inspection of the surface showed it to be a mixture of hard lumps and soft spots and it was quite uneven. There was also some block cracking of Section C near the door. A fine dusting of clay was applied and this was successfully worked into the cracks. Next, a tractor-mounted ripper was used to rip the subgrade surface in Section A to a depth of 130 mm. The loose material was then rotary-hoed to a fine tilth. The moisture content was then checked and found to be at the correct level. The subgrade surface was bladed level with a tractor blade and hand rakes. An attempt was made to use a 2 tonne tandem smooth steel drum roller, but two passes in static mode were enough to cause the material to start flaking. Also the surface was still a little uneven and the roller bridged the low spots so a rubber tyred roller trailer was used for a few passes. This produced soft spots that erupted at various locations in Sections A and B. That fact that Section B had soft spots was surprising, because this section was subjected to the most drying, because it was near the main entry doors. The soft areas were dug out with shovels. Some very soft and wet material was found at the bottom of Lift 5. This wet material was removed and replaced with drier material. Most of the digouts were small, but one at station 30 grew to nearly 1.2 m diameter. The removal and replacement of the material was conducted by hand and the compaction was done with the Ramex trench roller.

Insitu CBR testing conducted the next day showed that there was a hard crust in Section C. The finding of the crust confirmed that the method used to modify the moisture content of clay subgrades needs to be improved. The wheels of the CBR truck also exposed some more soft spots that were repaired. Other tests completed at this stage included density measurements with the NDM and sand replacement methods, Dynamic Cone Penetrometer, and Loadman tests. Laser level spot heights at primary stations were measured and manual profile beam readings were taken at each available station. The clay was completely covered by plastic sheets to stop drying until basecourse construction began.

Bison coils were placed at two depths at this point, as well as horizontal pairs at the surface. The horizontal pairs were placed by making a template for the circular saw so that two parallel cuts could be made in the clay surface 50 mm deep and 100 mm apart. The coils were pushed into the cuts and backfilled with fine clay. It took more than two days to place all the coils.

8.2.2.2 *Granular layers*

The granular layers were placed in two lifts. The first lift was 100 mm thick and the second lift was 150 mm thick.

A 100 mm thick layer of AP40 was placed as a sub-base layer in Section B, the ramp of the tank. As this lift was only 100 mm thick and placed directly on top of the subgrade, only one pass of the Wacker plate compactor was used to compact this lift. The only tests conducted on this material were Loadman tests at the primary sites. At this point in the construction process the Loadman had a problem with one of the internal rubber buffers and as the final lift of material was due to be placed the following day a decision was made to proceed without the completing the remaining Loadman readings on lift 6.

The initial lift thickness of the basecourse in Sections A and C was 100 mm. This was done to simplify the placement of the sub-base layer in Section B and to allow the placement of the Bison soil strain coils in the bottom 100 mm of the granular material. This lift was placed at the same time as the sub-base layer in Section B. This lift was compacted with one pass of the Wacker plate compactor. Loadman tests were conducted on the primary and secondary sites in Sections A and C on the intermediate lift (lift 6).

At this stage the final layer of Bison coils was placed in the basecourse.

The thickness of the final lift of the basecourse in all sections was 150 mm. The material was compacted with one pass of the Wacker plate compactor and the surface levels were then checked. The levels varied by up to 25 mm from the design levels. At this stage it was decided to use a small motor grader in an attempt to even out the surface. Even though the grader was the smallest machine available, it was still rather large for the test track. Working in a circle, the operator had no run-in or run-out area and in the first few passes far too much material was moved, making it worse than before the grader started. After two hours work the grader operator was able to get the levels back to about where they were prior to commencement. It was decided that any further attempts would not be productive. The initial levelling was conducted using the Bobcat loader and flat rakes. Over the next few days a combination of a 4 tonne drum/rubber wheel roller, the Wacker plate compactor, a small plate compactor, and a wobbly-wheeled trailer were used to compact and dress the basecourse surface. The dressing was carried out to present a tightly bound surface so that the surfacing layer could bind completely with the basecourse layer The Australian material was easier to work than the local AP20 M/4, requiring far less water to "fat it up".

The pavement was left for 10 days to dry out and then the surface was swept with a stiff broom to remove any loose material. The final surface tests included surface profiles, density testing by NDM and sand replacement methods and Loadman tests

8.2.2.3 *Asphalt surfacing*

The basecourse surface was heavily tack-coated with an emulsion primer. The surface layer was specified as a 25 mm thick layer of 5 mm asphalt. This material was placed by hand over the entire track. Compaction was conducted using a pedestrian roller. Manual surface profiles were measured and Loadman readings were taken on the sealed surface.

8.2.2.4 *As-constructed layer thicknesses*

The thicknesses of the various pavement layers are shown in figure 8.11. These values are taken from a single reading on the track centreline, at the transverse position of 1750 mm. A summary of the average values is shown in table 8.4. These values were obtained by averaging the readings over the trafficked width of 1000 mm. It can be seen that the thickness of the asphalt layer varied, with a difference of 11.3 mm between the minimum and maximum values, which was more than twice the maximum particle size (5 mm). The basecourse layer was more uniform, with a difference of 26.1 mm between the minimum and maximum values, which was closer to the maximum particle size of 20 mm. However the average thickness of 239.5 mm was 10 mm less than the design thickness of 250 mm.

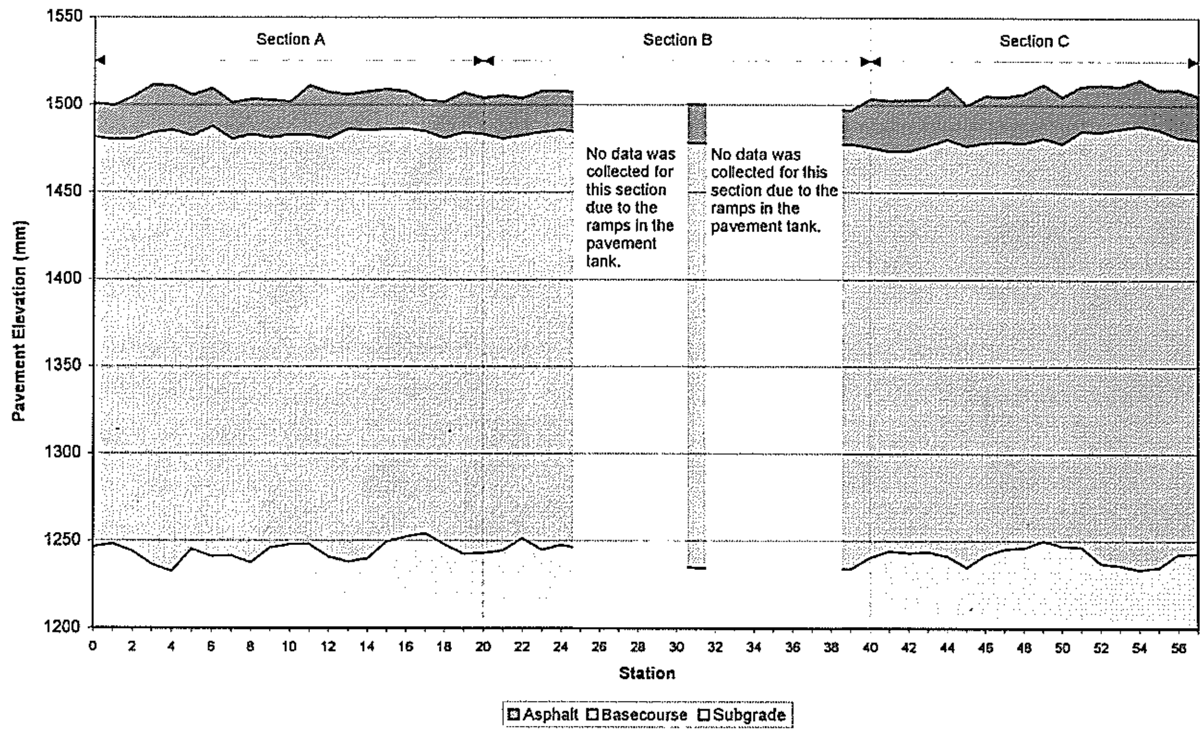

Figure 8.11 As-built longitudinal layer profiles along the pavement centreline

Table 8.4 Summary of layer thicknesses over trafficked pavement width

	Average over Trafficked Width (1000 mm) (mm)		
	Asphalt	Basecourse	Total Thickness
Average	23.7	239.5	263.2
Maximum	30.1	256.0	280.2
Minimum	18.8	229.9	248.7
Standard Deviation	2.7	6.6	6.8

8.2.3 Materials Properties

8.2.3.1 *Density and moisture content (subgrade)*

Density measurements were taken at the primary sites for the intermediate lifts of the subgrade material. On the final lift of the subgrade, both the primary and secondary sites were tested. In addition, on the final lift additional density measurements were taken at ± 0.5 and 1.0 m from the pavement centreline at the primary sites. All of these measurements were conducted using a Troxler Nuclear Density Meter (NDM). As a check on the accuracy of the NDM, sand replacement density tests were performed at the primary sites on the final lift of the subgrade material.

The results of the density testing on the subgrade are shown in figure 8.12 and table 8.5. It can be seen that the sand replacement values tended to be very similar to the values measured with the NDM.

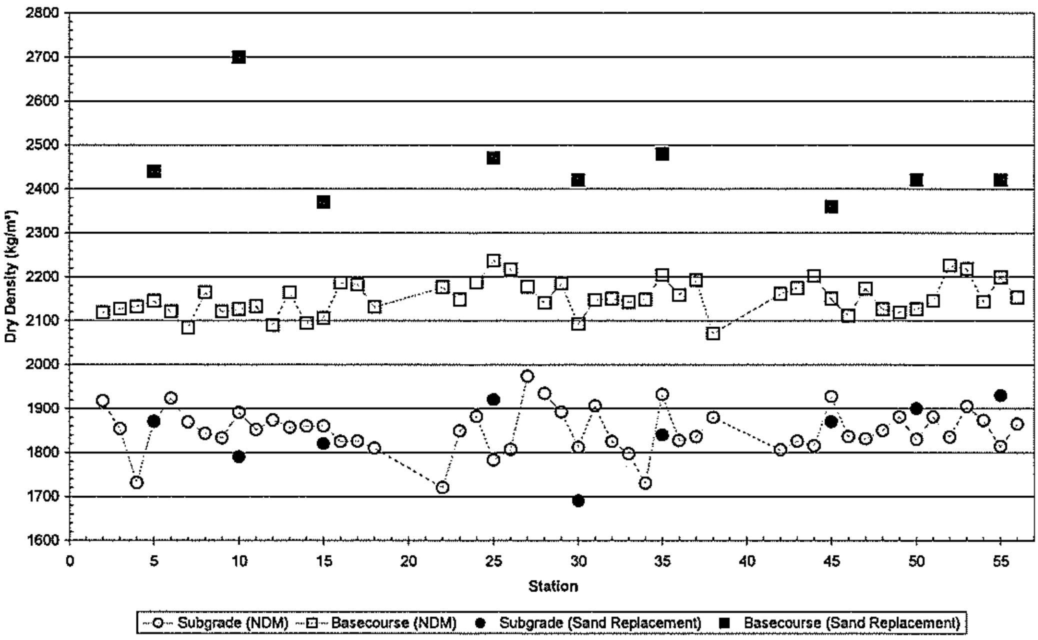

Figure 8.12 Density measurements on track centreline at top of subgrade and basecourse layers

Table 8.5 Density values for top of subgrade and basecourse layers

	Dry Density (kg/m³)					
Lift	No Rdgs	Average	Target	Minimum	Maximum	Standard Deviation
1	9	1786	1870	1679	1862	70
2	9	1765	1870	1694	1862	54
3	9	1845	1870	1811	1877	20
4	9	1861	1870	1810	1891	23
5 (NDM)	93	1852	1870	1720	1986	58
5 (Sand repl.)	9	1848	1870	1690	1930	75
7 (NDM)	93	2152	2100	2071	2237	39
7 (Sand repl.)	8[1]	2423	2100	2360	2480	42

[1] One value was excluded due to an obvious error.

During the placement of the material for each subgrade lift, the moisture content of the stockpiled material was measured with the microwave oven method. Once the moisture content was determined, the amount of water required to raise the material to the specified moisture was added to the surface of the material. Due to variations in the thickness and as-placed density of the material, the final moisture content tended to be variable. In future projects where the Waikari material is used, the moisture content should be more consistent throughout the material as it is mixed during the removal and replacement of the subgrade. The moisture content of the final lift of the subgrade is shown in figure 8.13. The readings were taken on the centreline of the track. The average values and the number of readings taken on each lift are shown in table 8.6. The moisture content values reported were determined with the nuclear density meter (NDM) and also by standard oven drying methods. The NDM consistently reported higher moisture content values than the values that were determined by oven drying.

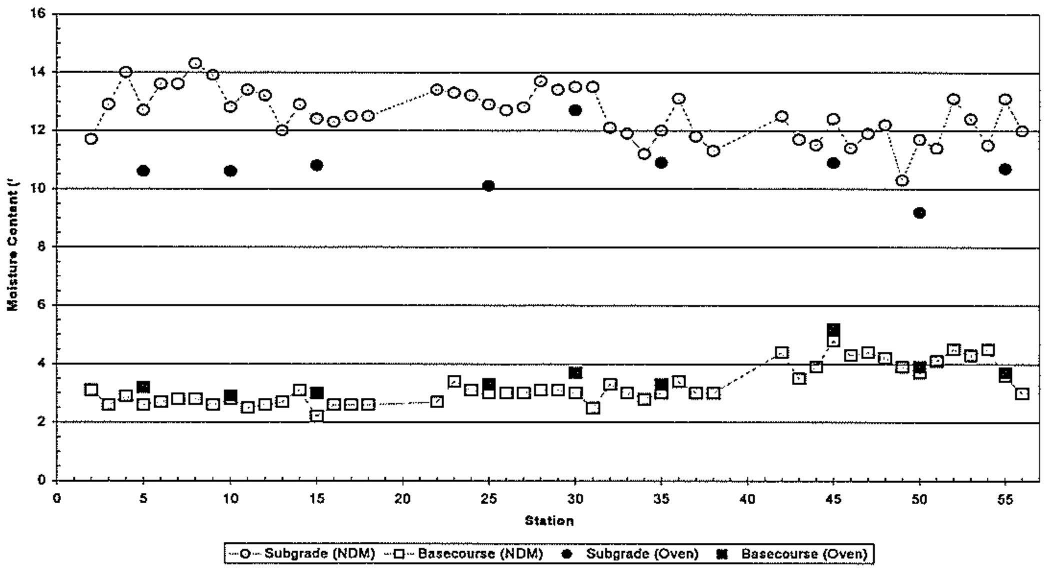

Figure 8.13 Moisture content of subgrade and basecourse materials along pavement centreline

Table 8.6 Moisture content values for top of subgrade and basecourse layers

Lift	No. of Readings	Average	Moisture Content (%)		
			Min. Reading	Max. Reading	Std Deviation
1	9	12.8	11.2	16.0	1.6
2	9	12.4	11.6	13.1	0.5
3	9	14.4	12.8	16.3	1.0
4	9	14.9	13.5	16.6	1.1
5 (NDM)	93	12.6	10.3	14.6	0.9
5 (Oven)	9	10.7	9.2	12.7	0.9
7 (NDM)	93	3.3	2.2	5.3	0.6
7 (Oven)	9	3.6	2.9	5.2	0.7

2.8.3.2 *Density and moisture content (granular material)*

Density measurements were taken at the primary and secondary sites for the final lifts of the basecourse material. In addition, on the final lift additional density measurements were taken at ± 0.5 and 1.0 m from the pavement centreline at the primary sites. All of the above measurements were made with a Troxler Nuclear Density Meter (NDM). As a check on the accuracy of the NDM, sand replacement density tests were performed at the primary sites on the final lift of the basecourse material. The sand replacement values tended to be greater than the values measured with the NDM. Both the NDM and sand replacement values are shown in figure 8.11 and table 8.5. The sand replacement density value reported for Station 10 (2700 kg/m^3) would appear to be in error, as the solid density of the parent material is only approximately 2650 kg/m^3. This result was left in for completeness only. The statistical values were calculated without this value. The remaining sand replacement values show a low level of variation, but they are consistently 2-300 kg/m^3 higher than the values measured with the NDM. One possible explanation is that the NDM values are affected by the condition of the surface of the material being measured and that the surface contained enough irregularities to affect the readings.

The moisture content of the final lift of the basecourse is shown in figure 8.13. The readings were taken on the centreline of the track. The average values and the number of readings taken on each lift are shown in table 8.6. The moisture content values reported were determined with the nuclear density meter (NDM) and also by standard oven drying methods.

8.2.3.3 *Loadman*

The Loadman was used extensively during the construction to monitor the strength of the material as the Nuclear Density Meter was proving to be inconsistent with the readings in the pavement tank. Each of the Primary sites was tested on each lift. All of the primary and secondary sites were tested for the final lift. The results of the Loadman measurements for the top of the subgrade and basecourse layers are shown in figure 8.14 and table 8.7, whilst a summary of the Loadman results for each section, including the number of readings taken, is shown in table 8.8.

During the construction period, the modulus values that were reported by the Loadman were used to provide an indicator of the consistency of the compaction, rather than a measure of absolute value. Other researchers have reported on the shortcomings of the calculation algorithms used by the Loadman manufacturers (Bartley Consultants Ltd. 1998).

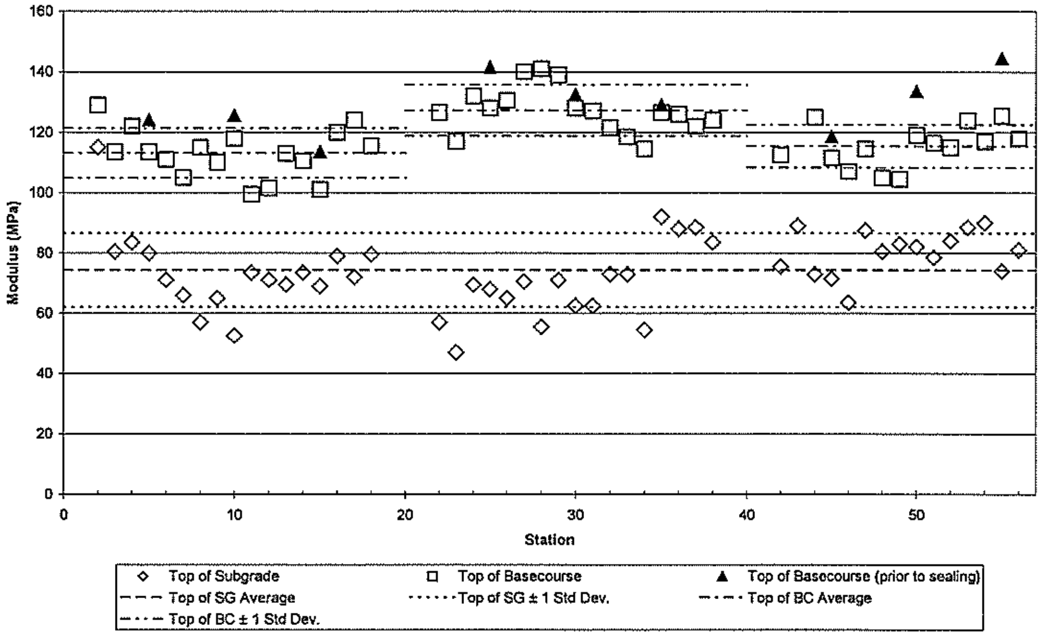

Figure 8.14 **Loadman modulus values along pavement centreline for top of subgrade and basecourse**

Table 8.7 Loadman modulus values for top of subgrade and basecourse layers

			Loadman E Modulus (MPa)		
Lift	No. of Readings	Average	Min. Reading	Max. Reading	Std Deviation
1	9	56	32	77	13.3
2	9	57	51	72	6.3
3	9	47	35	61	7.5
4	9	45	28	58	10.3
5	93	75	45	115	12.2
6	39	81	62	100	11.2
7	93	118	100	141	8.9
7a	9	154	142	170	10.1

Table 8.8 Summary of Loadman E values for each section

	No. Readings	Loadman E Values (MPa)			
		Average	Maximum	Minimum	Std Dev.
	Subgrade				
TNZ AP20	17	74	115	53	13.4
AP40/TNZ AP20	17	69	92	47	12.8
Australian AP20	15	80	90	64	7.5
	Basecourse				
TNZ AP20	17	113	129	100	8.2
AP40/TNZ AP20	17	127	141	115	7.7
Australian AP20	15	116	126	105	7.1

8.2.3.4 *Insitu CBR*

Insitu CBR testing was undertaken at the primary sites on the top of the final lift of subgrade material. The results of these tests are shown in table 8.9. The results were higher than expected, although the surface of the subgrade material had a thin, but noticeable, crust on it. A hole was augered through the crust and an additional CBR test was performed at a depth of 60 mm below the surface. The result from the surface test was 40%, whereas the result from the test performed below the crust was 14%, approximately one-third of the surface result.

Figure 8.15 compares the results of the Loadman and insitu CBR test results conducted on the top subgrade layer. It can be seen that there was a reasonable correlation between the two devices.

Table 8.9 Insitu CBR test results on subgrade surface after construction

Station	Test Location	CBR value (%)	
		at 2.5 mm pen	at 5.0 mm pen
5	surface	35	27
10	surface	11.7	11.9
15	surface	22.3	20.2
25	surface	13.7	12.5
30	surface	10.5	13.5
35	surface	44.7	38
45	surface	39.4	NR[2]
45	-60 mm	14[1]	14.5[1]
50	surface	49	NR[2]
55	surface	35.6	NR[2]

Tests were performed with an 5.0 kg surcharge (diameter of surcharge weight is 150 mm).

[1] no surcharge (test done in 100 mm diameter hole).

[2] These results were in excess of 50.

8.2.3.5 *Dynamic cone penetrometer*

Dynamic cone penetrometer tests were undertaken at the primary sites on the top of the final lift of subgrade material. The results of these tests are shown in table 8.10. The e values (mm/blow) have been converted into CBR values using figure 5.2 from the Austroads Pavement Design Guide (Austroads 1992). The inferred values were determined by interpolating the number of blows required to penetrate through the top 100 mm of the subgrade material. This value was then converted into an average mm/blow value. The inferred CBR values were then calculated with this mm/blow value. Therefore these values represent an average CBR of the top 100 mm of the material while the CBR values determined from the in-situ test represent the strength of the surface. The apparent crust that formed on the top of the material has overstated the 'average' strength of the material. If the test results from station 45 are analysed, the test conducted at a depth of 60 mm gave a CBR value of 14, compared with 39 for the surface test. From this result, it could be concluded that the average CBR value of the top 100 mm of the subgrade would be much lower than the reported values, mainly due to the crust on the surface. In future experiments, the in-situ CBR tests should be done at a depth of 50 mm below the surface in order to remove the effect of a crust, if one has formed.

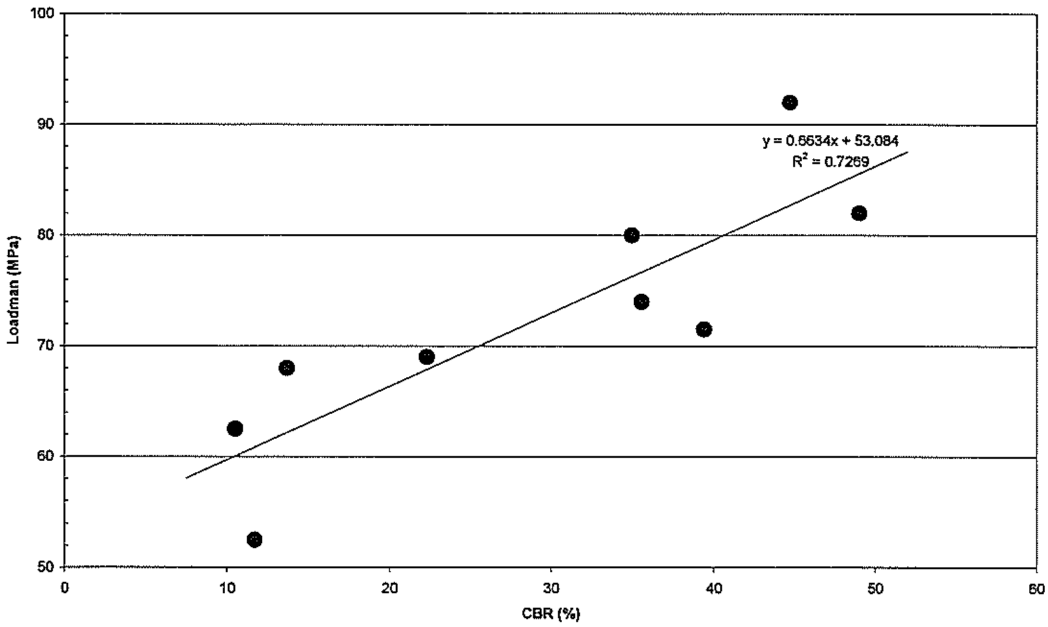

Figure 8.15 Correlation between the Loadman and in-situ CBR test results for the subgrade

Table 8.10 Insitu and inferred CBR values from top of subgrade

		CBR (%)		
Section	**Station**	**Insitu**	**Inferred from DCP**	**Average**
A	5	35	16	11
	10	12	9	
	15	22	8	
B	25	14	11	15
	30	11	9	
	35	45	25	
C	45	39		17
	45 (-60mm)	14	15	
	50	49	18	
	55	36	17	

8.2.4 Results of Zero Testing

8.2.4.1 *As Built Structural Capacity*

Prior to trafficking, CAPTIF Deflectometer measurements were carried out at the primary and secondary test sites. The results are shown in figure 8.16, whilst a summary of the statistical values is shown in table 8.11. It can be seen that there was a low negative correlation between the Loadman results from the basecourse and the deflections (–0.48). When the different material strength measurements (inferred/DCP CBR, Loadman and Deflectometer) are separated into the three pavement sections, there are differences in each section.

The Loadman results on the subgrade show that the order of the three sections, from lowest to highest average is B, C, A. However, the variation in the individual results for Section C is lower than the other sections. The Loadman results for the basecourse are different: Section B has the highest Loadman values, while Sections A and C have similar values. One possible reason for this is that the construction traffic has to pass over this section when entering and exiting the pavement tank; therefore the compactive effort is greater than for the other sections. The inferred CBR values from the DCP testing (table 8.10) suggest that Sections B and C have an average inferred CBR of 15 and 17 respectively, while the value for Section A is 11%.

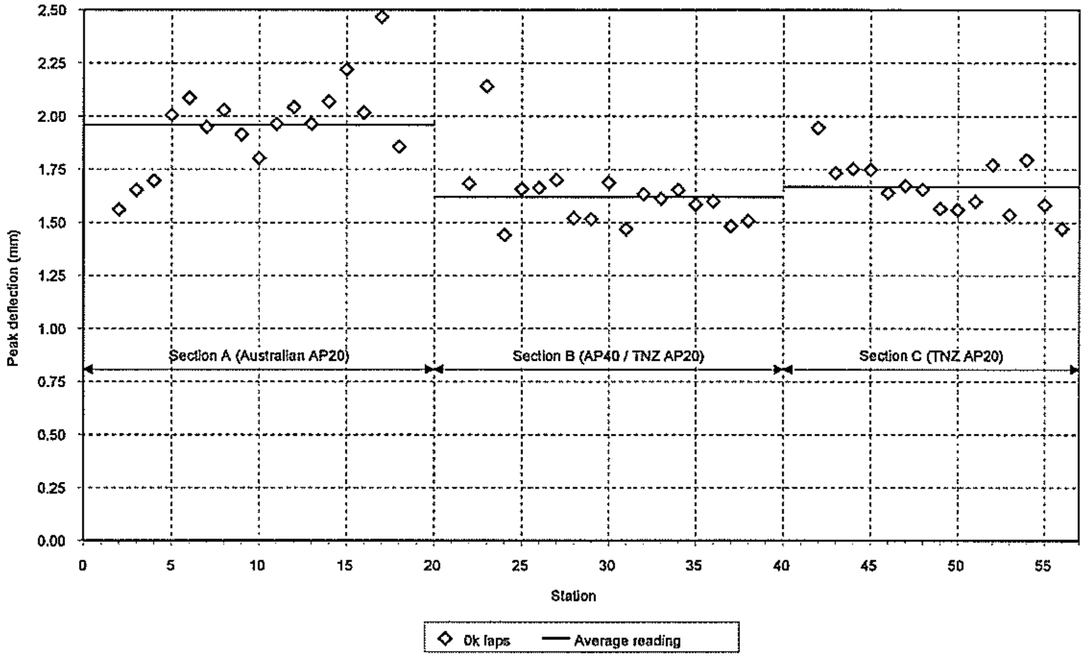

Figure 8.16 Peak CAPTIF Deflectometer values at 0 cycles

Table 8.11 Summary of CAPTIF Deflectometer Values at 0k Laps

	Peak CAPTIF Deflectometer Value (mm)				
	No. Readings	**Average**	**Maximum**	**Minimum**	**Std Dev.**
TNZ AP20	17	1.958	2.466	1.560	0.213
AP40/TNZ AP20	17	1.620	2.141	1.441	0.158
Australian AP20	15	1.668	1.945	1.471	0.123

Analysis of the deflection results in figure 8.16 suggests that Sections B and C had a similar initial strength, while the strength of Section A was lower. From the above discussion, it could be postulated that the strength of the subgrade influenced the CAPTIF Deflectometer results.

8.2.4.1 *As-constructed longitudinal profile*

Results from the Laser Profilometer are shown in figure 8.17. The absolute levels of the pavement centreline have a vertical range of 20.5 mm.

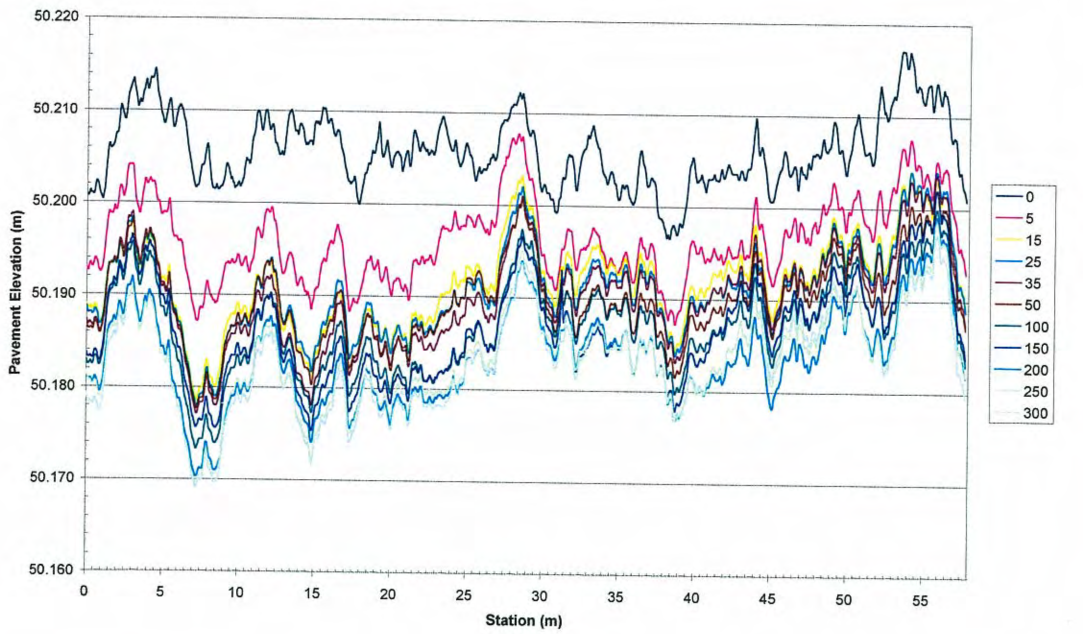

Figure 8.17 Longitudinal profiles for the track centreline (transverse position = 175 cm)

8.2.5 Accelerated Pavement Loading

After the completion of the CAPTIF Deflectometer testing on the as-built pavement, 5,000 load cycles were applied to the pavement. This was conducted with a rectangular distribution over the entire trafficable width of the pavement. This corresponded to a ram/transverse position of 175±50 cm. The 5000 load cycles corresponded to 10,000 ESAs.

Additional testing was conducted after 5,000 load cycles to determine the effect of lateral position of the vehicles on pavement sub-surface response. For the remainder of the accelerated loading the vehicle wander was set to a normal distribution with a centreline value of 1750 mm and a standard deviation value of 100 mm. This gave an effective trafficked width of 900 mm (500 mm for the width of the dual tyre footprint and 400 mm for the vehicle wander).

Accelerated loading was halted after 15, 25, 35, 50, 75, 100, 150, 200, 250 and 300 kcycle to allow for intermediate testing. At each of these load cycle counts the following tests were performed:

- transverse profiles at each station;
- longitudinal profiles at ram positions of 095, 135, 175, 215 and 255 cm;
- bison soil strain measurements; and
- dynamic wheel force measurements.

In addition to these tests, CAPTIF Deflectometer readings were taken at the primary and secondary test sites after 15, 50, 150, 250 and 300 kcycle.

8.2.6 Results of Testing

8.2.6.1 *Longitudinal profiles*

Longitudinal profiles from selected measurement intervals for a transverse position of 1750 mm are shown in figure 8.18, together with the differences in the profiles between 0 and 5 and 15 and 25 kcycle. Examination of this difference data shows that the change in profile between 0 and 15 kcycle and 0 and 25 kcycle was very similar. From this it can be concluded that the post construction consolidation has been completed after 15 kcycle. The difference between the profiles at 0 and 15 kcycle and 15 and 300 kcycle is shown in figure 8.19. This shows that the change in the longitudinal profile in the first 15 kcycle was quite high in Section A and similar in Sections B and C. The change in profile between 15 and 300 kcycle was similar in all three sections (8.0 mm).

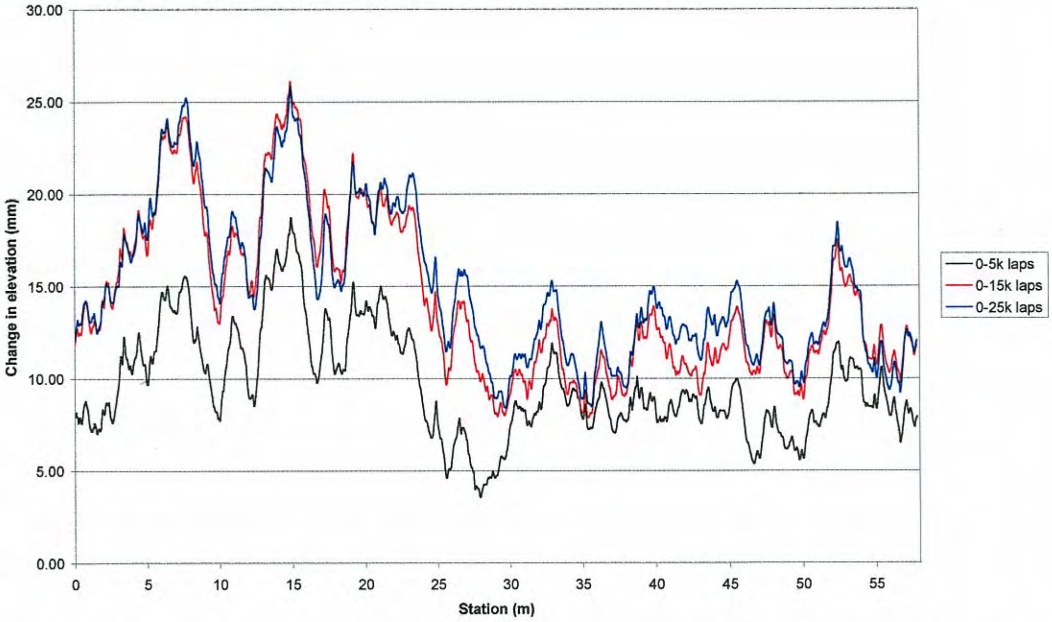

Figure 8.18 Change in longitudinal profile between 0 and 5, 15 and 25 kcycle for the track centreline (transverse position = 175 cm)

The initial change in the longitudinal profile mirrors the peak pavement deflection measurements at 0 cycles, with higher deflections being recorded in Section A compared with Sections B & C, and this behaviour is also apparent in the initial changes in profile.

Analysis of the profiles that were measured beyond the trafficked area show that there was some upwards movement at locations around the track in the three periods investigated (0-15, 15-300 and 0-300k laps). The average values for the changes in profiles are shown in table 8.12. The maximum upwards movement in the first 15 kcycle was less than 5 mm, with average values ranging from -1.7 to 0.4 mm. The changes from 15-300 kcycle were mostly positive, i.e. downwards movement of the pavement surface. The conclusion from this analysis is that there was very little shoving and/or heaving of the pavement surface.

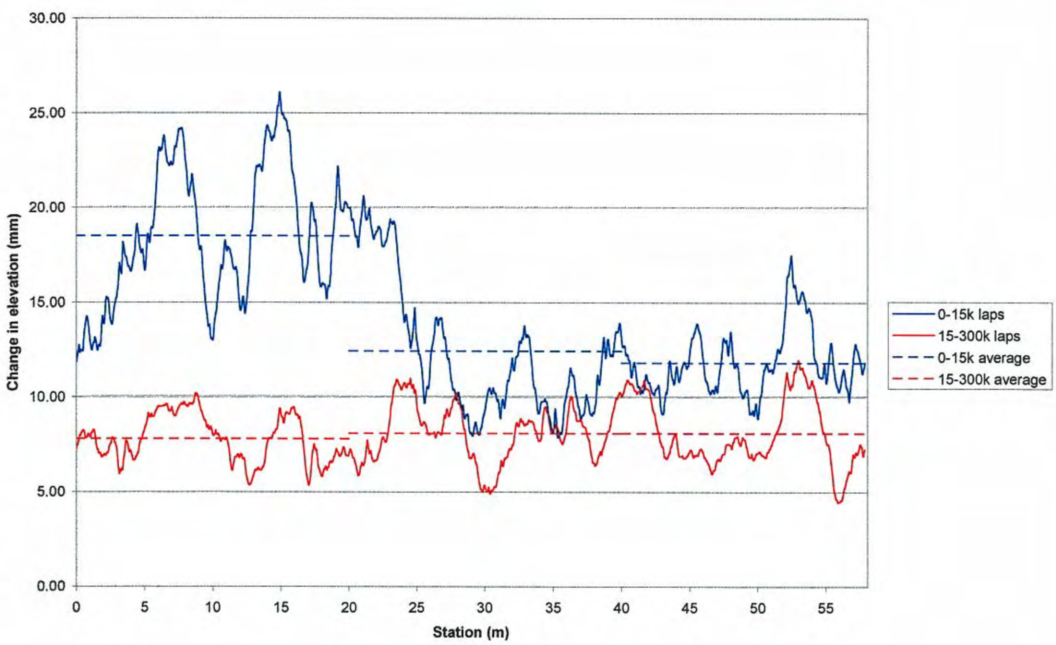

Figure 8.19 Difference between 0 and 15k and 15k and 300 kcycle for the pavement deformation from the track centreline (transverse position = 175 cm)

Table 8.12 Statistical Values for Difference in Longitudinal Profiles for Track Centreline Between 0 and 15k and 15k and 300 kcycle

	Station		
	0-20	20-40	40-57.8
0-15 kcycle			
Average	18.5	12.4	11.8
Standard Deviation	3.6	3.6	1.7
Minimum	11.8	7.9	8.9
Maximum	26.1	20.6	17.5
15-300 kcycle			
Average	7.8	8.1	8.1
Standard Deviation	1.2	1.5	1.8
Minimum	5.4	4.9	4.4
Maximum	10.2	11.0	12.0

The longitudinal profiles were used to calculate the International Roughness Index (IRI) for the pavement. However, the specification for calculating the IRI specifies a simulated speed for the quarter car model of 80 km/h and a sample length of 200 m. Because the profile length is only 58 m at CAPTIF, the IRI values obtained from CAPTIF data have a degree of uncertainty. Also because of the construction techniques used at CAPTIF, there tends to be a high number of short wavelengths in the pavement profile which affect the calculation of the IRI.

8.2.6.2 *Transverse Profiles*

Transverse profiles were taken at each station for each measurement interval. From the data collected two parameters were calculated for each station, viz. the vertical deformation and a simulated rut depth. The deformation is the difference between the pavement elevation at a transverse position of 1750 mm at the current lap count and the same position at 0 laps. The simulated rut depth is obtained by placing a virtual straight edge across the pavement profile and determining the maximum distance between it and the pavement profile. The algorithm that calculates this parameter determines the highest point on either side of the track centreline, then the slope and intercept of the line that crosses between the two high points is calculated. The program then iterates through the profile points between the two high spots and finds the maximum value between the simulated line and the pavement elevation.

The average values of the vertical deformation for the three sections is shown in figure 8.20 whilst the average values of the straight edge rut depths for the three sections is shown in figure 8.21. In both figures, irregularities in the average point measured is caused by the effect of the build-up of rubber on the track from the tyres. Although the larger lumps of rubber were removed prior to measurements being taken, some of the smaller patches were unable to be removed. This has led to as increase in the pavement height at the point of measurement.

The measured transverse profiles for Stations 4, 14, 24, 31, 51, 54 are shown in figures A1 to A6 in appendix A. These stations were selected as they are the locations of the post-mortem trenches. It can be seen that the location of the lowest point in the rut was 150-200 mm outside of the centreline of the track. The locations of the transverse points were checked when this shift became apparent and the only explanation found was that changes in vehicle geometry to a the shift in the position of the maximum rut (the vehicle camber was increased to the maximum negative amount in order to counter some of the dynamic aspects of the vehicle performance).

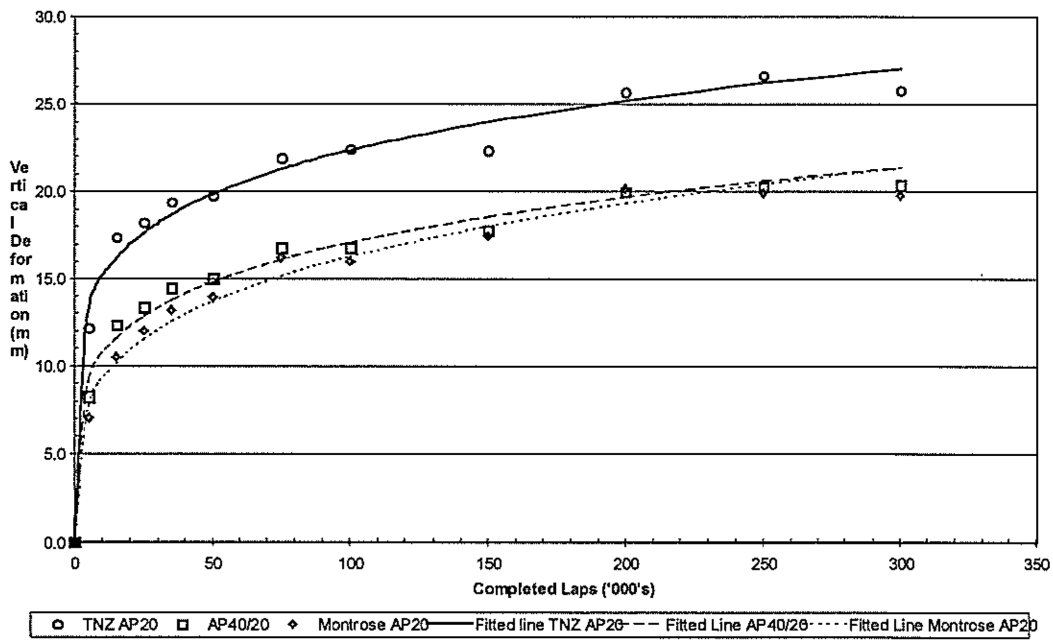

Figure 8.20 Average vertical deformation for the three test sections

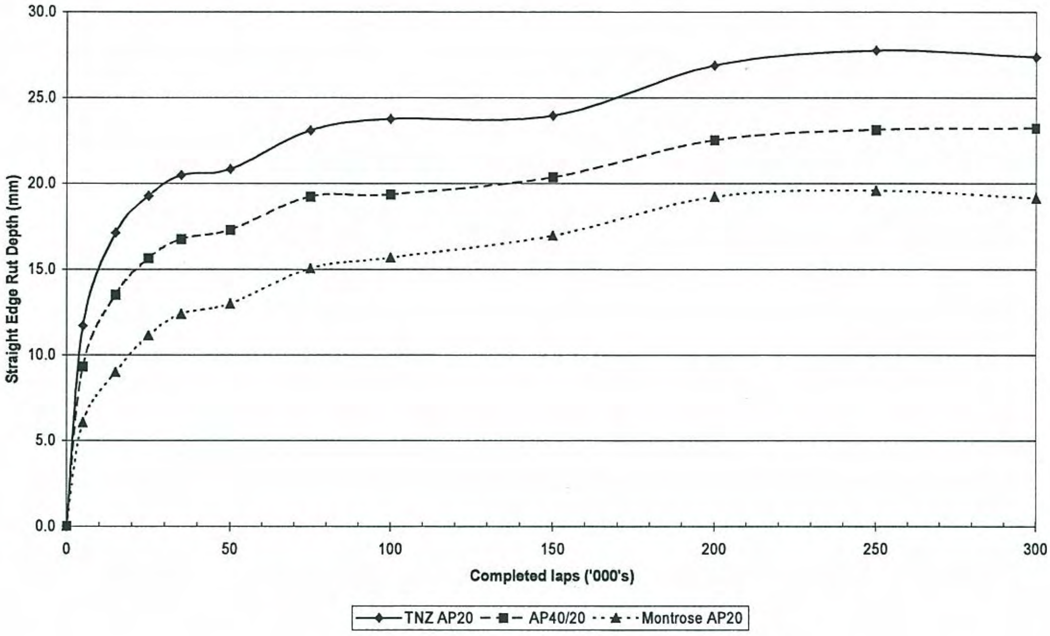

Figure 8.21 Average straight edge rut depths for each section

The progression of vertical deformation and straight edge rut depths for the six stations listed previously can be seen in figures 8.22 and 8.23 respectively. Analysis of the rutting data shows that the rate of rutting was high in the first 35,000 cycles, with the incremental rate of rutting falling below 0.10 mm per 1,000 cycles after 35,000 cycles (figure 8.24). There was no significant difference in the rate of rutting between the three sections after 25,000 cycles.

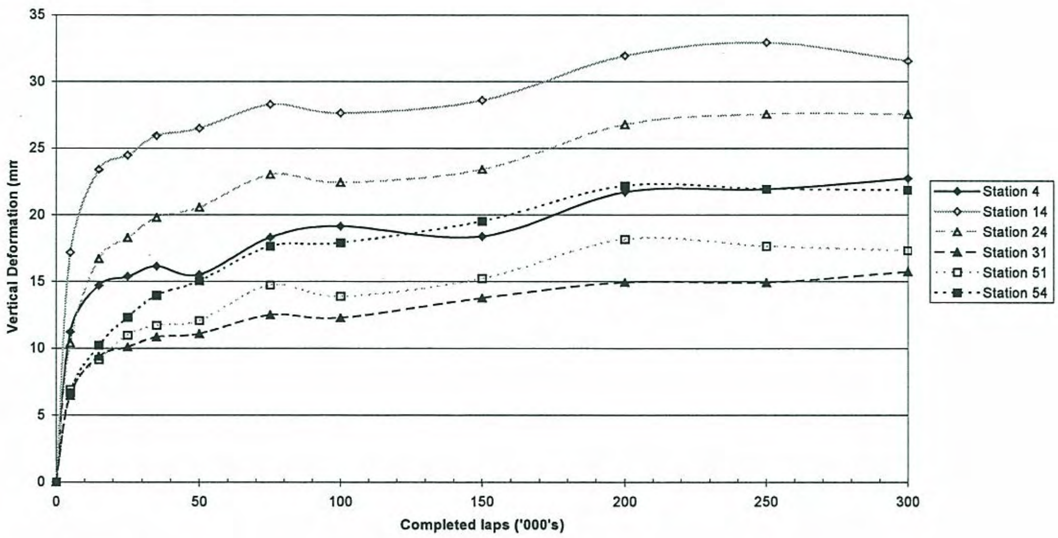

Figure 8.22 Vertical deformations for the post-mortem stations

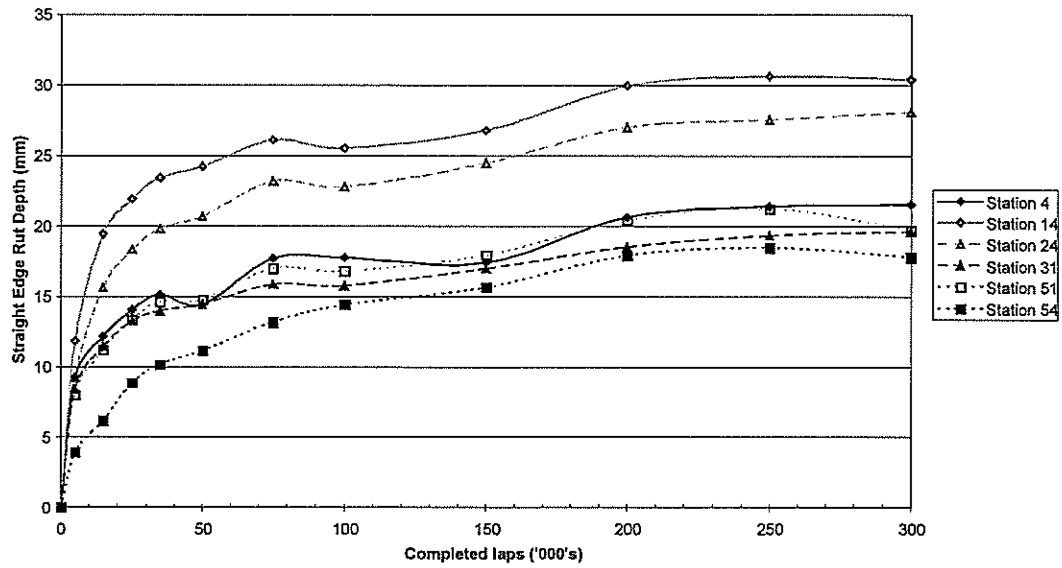

Figure 8.23 Straight edge rut depths for the post-mortem stations

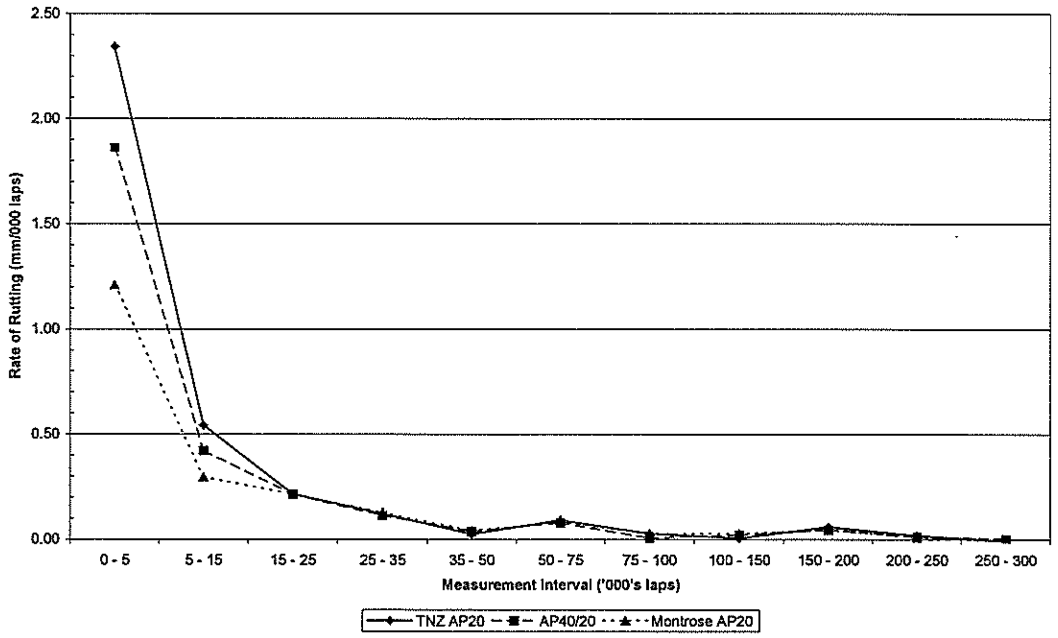

Figure 8.24 Incremental rate of rutting for the three sections

8.2.6.3 *Pavement Deflections*

Pavement deflections were measured at the primary and secondary sites after 15k, 50k, 150k, 250k and 300k cycle. The average values for these measurements are shown in table 8.13 and figure 8.25, whilst the deflections for the six post-mortem stations are shown in figure 8.26. As with all of the data presented so far in this report, the values increased rapidly in the first 50 kcycle and then remained reasonably constant to the end of the test. The deflection bowl shape is consistent for all of the measurements. A typical deflection bowl is shown in figure 8.27. As can be seen from the shape of the bowl, the bowl was very short and almost all of the deflections occurs within 1.0 m of the wheel. The shape of the bowl makes it very difficult to back-calculate the modulus values of the different pavement layers.

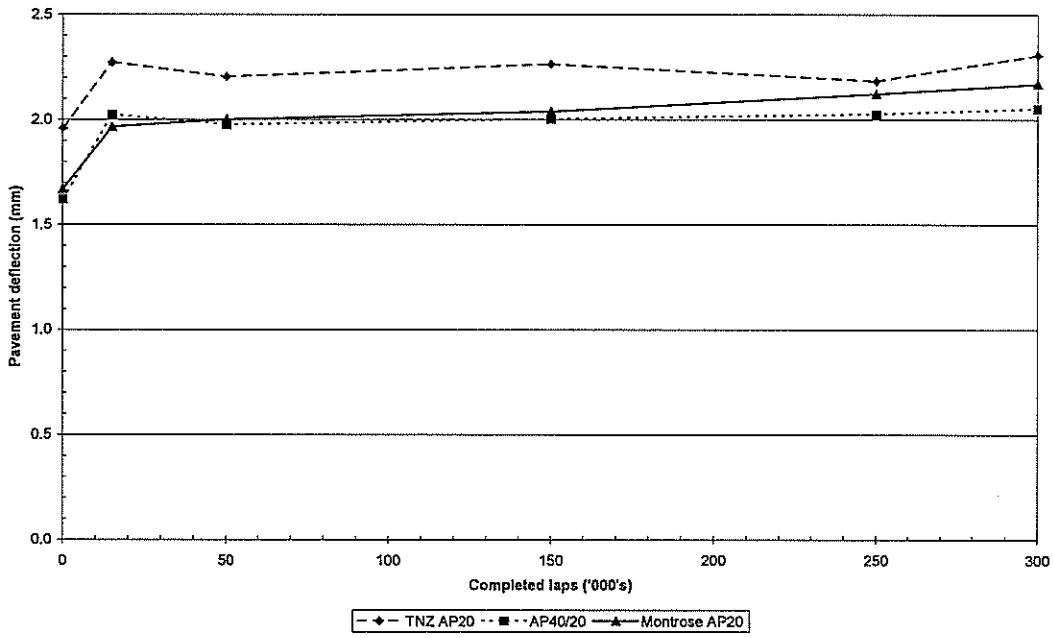

Figure 8.25 Average deflections during testing.

Table 8.13 Peak deflection readings during testing.

	Peak CAPTIF Deflectometer Readings (mm)					
Completed Cycles ('000's)	**0**	**15**	**50**	**150**	**250**	**300**
TNZ AP20	1.958	2.272	2.203	2.263	2.183	2.306
AP40/20	1.620	2.023	1.975	2.003	2.026	2.052
Australian AP20	1.668	1.966	2.001	2.037	2.121	2.169

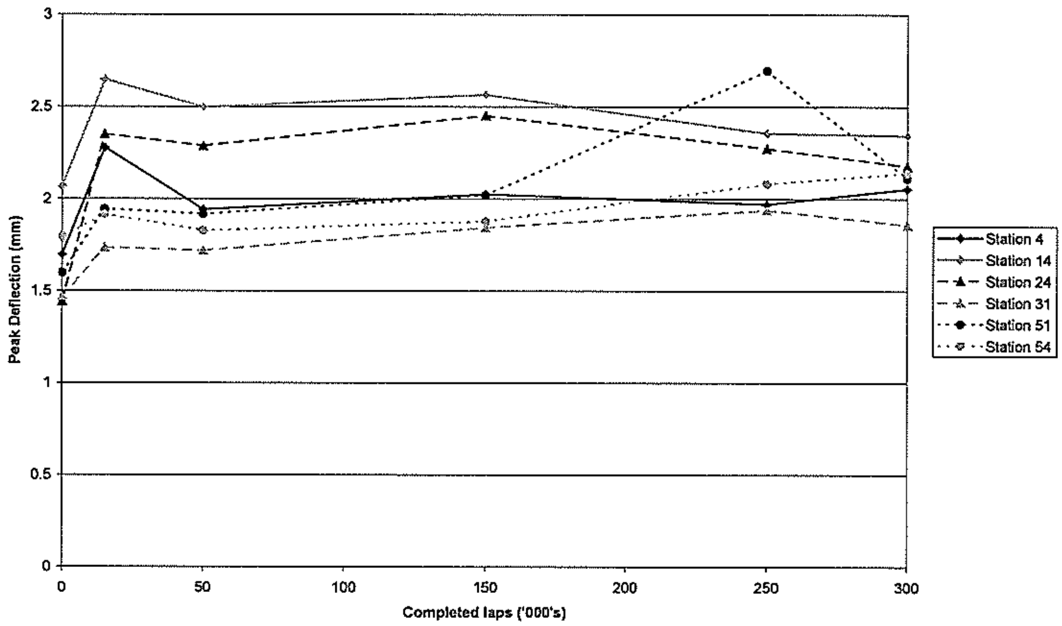

Figure 8.26: Deflectometer readings for the post-mortem stations.

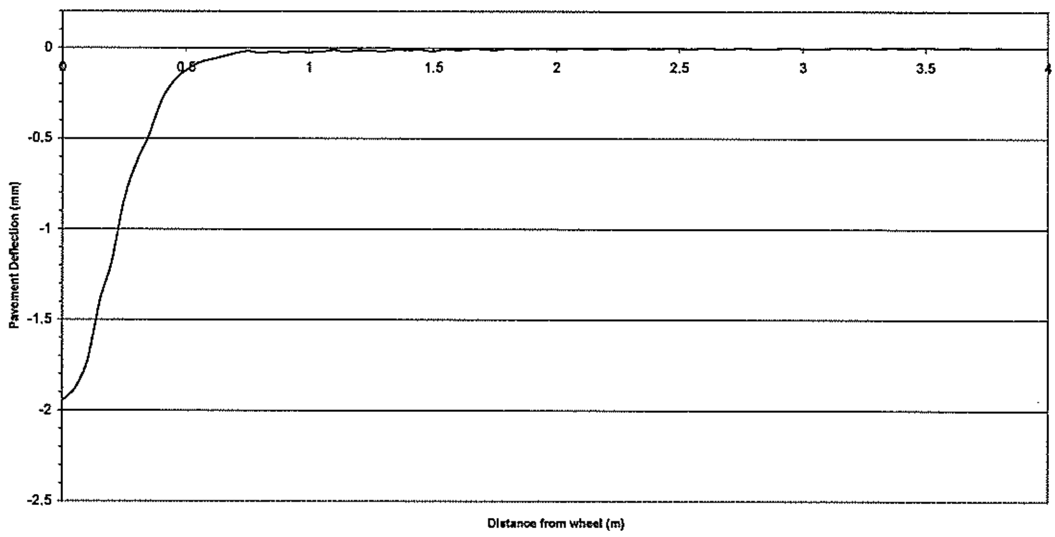

Figure 8.27 Deflectometer reading for Station 56-175 at 300 kcycle.

8.2.6.3 *Bison soil strains*

During this project the Bison strain coils suffered from a durability problem, with 17 out of the 24 coil pairs failing. All of the failed pairs were in the upper section of the pavement, either in the basecourse layer or the top of the subgrade. The pairs in the top of the subgrade have one of the coils located at the interface of the two layers. It is thought that the high initial deformations in the granular layers either damaged the coils/cables or caused the response to go out of the range of the measuring equipment. The responses of the coils at 45 km/h for sections A and C are shown in f

figures 36 and 37 respectively. The Upper Subgrade (1) location refers to strains in the top 100 mm of the subgrade layer, whilst the Subgrade 2, 3 and 4 measurements refer to measurements 100-200 mm, 200-300 and 300-400 mm below the surface of the subgrade respectively.

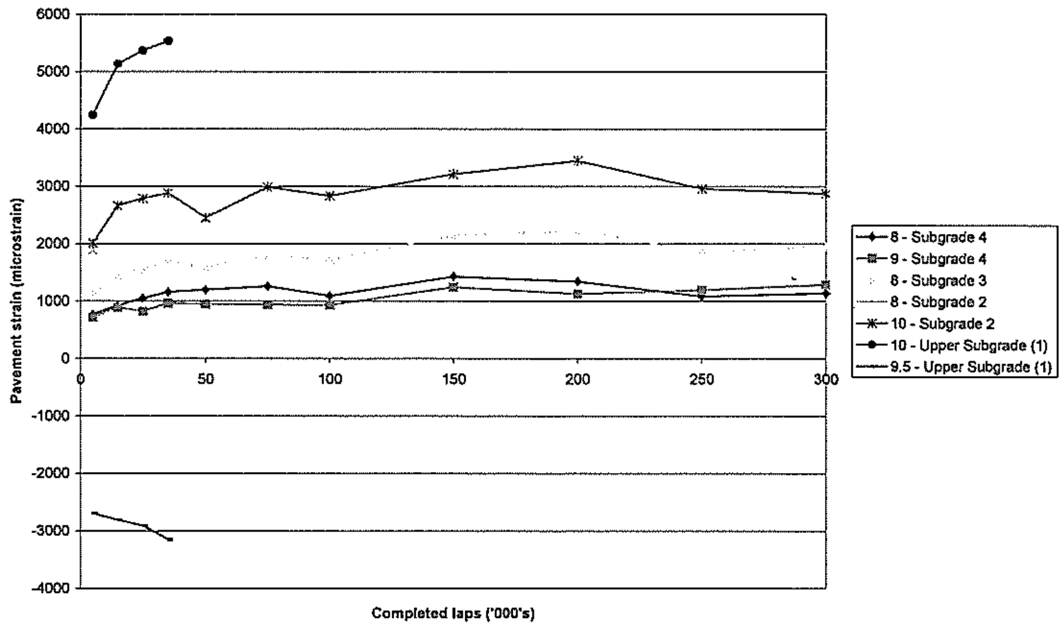

Figure 8.28 Bison soil strain measurements for Section A (TNZ AP20 material).

Overall the strains in Section A tended to be higher than those in Section C. One possible reason could be that Section A was slightly weaker than the other sections, as suggested from the measurements taken during and after pavement construction.

This project was the first time that strain measurements were recorded deeper than 200 mm below the surface of the subgrade. The average strains at each depth are shown in table 8.14. If there was more than one station in each section at the same depth, then all of the stations were included in the averaged values. It is not completely correct to average and compare between the different stations in each section as variations between the stations will arise due to differences in the structural capacity of the pavement and differences in the dynamic wheel forces between the stations. Analysis of the averaged results shows some similarities between the two sections, namely the ratio between the strain in the lower basecourse and Subgrade 1 Level was similar for Sections A (2.79) and C (2.76). The other interlayer ratios showed more variation with the differences between the two sections ranging from 0.22 to 0.06.

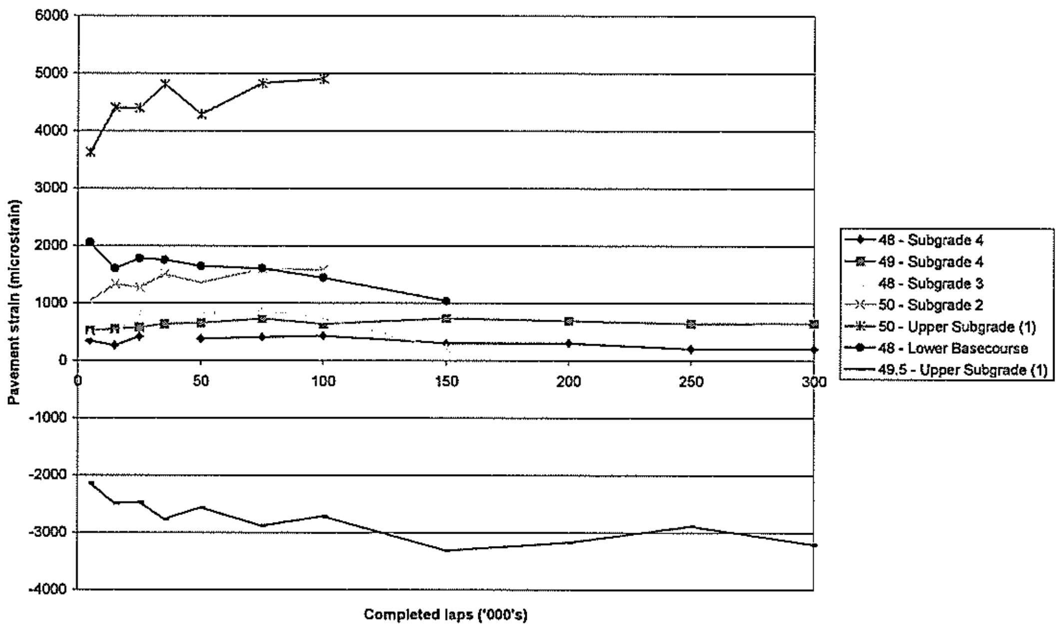

Figure 8.29 Bison soil strain measurements for Section C (Australian AP20 material).

Table 8.14 Average pavement strains for the two test sections.

	Vertical Compressive Pavement Strain (microstrain)	
	Section A: TNZ AP20	**Section C: Australian AP20**
Basecourse	2205	1615
Subgrade 1	6153	4464
Subgrade 2	2827	1382
Subgrade 3	1734	761
Subgrade 4	1067	637

8.2.7 Post-Mortem

After the completion of the accelerated loading stage, six stations were chosen for the post-mortem trenches. Two stations were chosen from each section, with one station corresponding to the location of minimum rut depth and the other station corresponding to the location of maximum rutting. The selected stations and the selection criteria are shown in table 8.15. The selected stations were not necessarily the maximum or minimum values, but close to the required values.

Table 8.15 Post-mortem stations and selection criteria.

Section	Station	Selection Criteria
A: TNZ AP20	4	Minimum
	14	Maximum
B: AP40/20	24	Maximum
	31	Minimum
C: Australian AP20	51	Minimum
	54	Maximum

At each location manual profile measurements were taken on the top of the surface prior to removal of the surfacing material. Manual profile measurements were repeated on top of the basecourse layer and then density measurements (sand replacement and nuclear) and Loadman test results were carried out. An excavator with a narrow bucket (300 mm) was then used to remove the basecourse material to a point just above the subgrade interface and the remaining material was removed by hand.

Once the subgrade surface had been exposed the manual profile, density and Loadman tests were repeated. In addition Dynamic Cone Penetrometer tests were also performed on the surface of the subgrade.

8.2.7.1 *Permanent Deformation*

The layer interfaces from construction and at the end of testing are shown in figures B1 to B6 in appendix B, whilst a summary of the changes in the thicknesses of the different pavement layers is shown in table 8.16. These values are average values across the trafficked width of pavement. From this table it can be seen that the change in thickness of the asphalt layer was negligible and that the magnitude of the rutting in the subgrade was similar for all of the stations (3.5 mm) except Station 14 (6.6 mm). However, a small amount of punching of the aggregate into the subgrade layer was observed in the trenches. The majority of the rutting occurred in the granular layers, which was most probably due to insufficient compaction during the construction of the pavement. Similar behaviour was, however, observed in the ALF benchmark trial (Vuong and Sharp 1997).

Table 8.16 Changes in pavement layer thicknesses.

	Change in Layer Thicknesses (mm)				
Station	Δ AC	Δ BC	Δ SG	Δ Pavement Thickness	Surface Vertical Deformation
4	0.3	10.6	3.1	14.0	17.1
14	-0.1	11.4	6.6	17.9	24.5
24	1.2	14.2	3.0	18.4	21.4
31	0.4	6.4	2.7	9.5	12.2
51	0.8	7.0	2.9	10.7	13.6
54	0.7	10.2	2.6	13.5	16.1

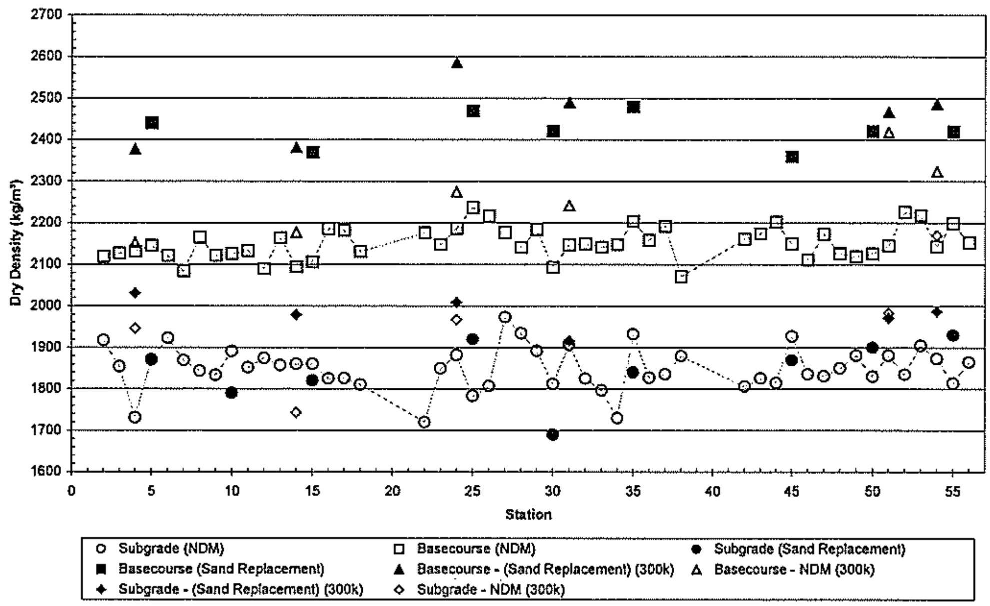

Figure 8.30 **Sand replacement and NDM densities after 0 and 300 kycle.**

8.2.7.2 *Density*

The densities from the post-mortem testing are shown in figure 8.30. Overall, there was a slight increase from the construction values for the basecourse and subgrade materials. There was, however, still a difference between the densities obtained by the sand replacement and nuclear methods. One possible reason for the high (2-300 kg/m^3) difference is that the technique and/or calibration of the sand replacement device needs to be checked. The densities determined by the sand replacement method are higher than expected, given the large amount of densification/compaction observed in the granular layer.

Table 8.17 **Loadman Modulus values before and after testing.**

	Subgrade			Basecourse		
Station	**0k**	**300k**	**% change**	**0k**	**300k**	**% change**
4	84	58	-30.5	122	181	48.4
14	74	70	-4.8	111	159	43.9
24	70	57.5	-17.3	132	168.5	27.7
31	63	69	10.4	127	162.5	28.0
51	79	59.5	-24.2	116.5	160	37.3
54	90	62.5	-30.6	117	182	55.6
Average	76	63	-16	121	169	40

8.2.7.3 *Loadman*

The Loadman modulus values from before and after the test are shown in table 8.17. The subgrade values actually decreased during the test, although the average decrease was only 16 MPa, which was almost within the repeatability of the device (10 MPa). The basecourse value increased by, on average, 40%. This finding is in agreement with the other test data which showed that the basecourse densified/increased in strength during trafficking.

9. Conclusions

A comprehensive review of the CAPTIF and ALF facilities was carried out and the findings are presented in sections 2 and 3 respectively. A detailed comparison of the facilities is provided in section 7.

9.1 Review of CAPTIF and ALF Facilities

The major differences between the CAPTIF and ALF facilities are the type of loading (circular v. linear); the maximum speed of loading (45 km/h v. 20 km/h); the length of test section (58 m v. 12 m); the environment controls (fully enclosed pavement tank and test building, no temperature control v natural pavement and outdoors, limited temperature control); and the location of tests (permanent location v mobile facility). Both facilities have access to comprehensive testing equipment and testing laboratories which allows the condition of test pavements to be monitored in detail during construction and loading. In past projects, ALF data acquisition equipment has been used in CAPTIF tests and vice versa.

CAPTIF is ideal for 'ranking' studies because up to three pavements can be tested at the same time and, if necessary, two magnitudes of load. This makes the facility ideal for load equivalency investigations, determining the effect on the pavement of different tyre types and pressures and measuring the effect of vehicle dynamics on the performance of the pavement. The indoor facility is ideal for conducting fundamental studies, where the number of outside influences that could effect the outcome of a trial can either be minimised or measured. ALF is more suited to 'construction/rehabilitation' studies as pavements can be constructed with specialised road construction plant and techniques.

Personnel involved in the management and operation of the two facilities regularly exchange ideas and information on test programmes and test methods in order to maximise the returns from both facilities.

9.2 Review of CAPTIF and ALF Test Programmes

A comprehensive review of the CAPTIF and ALF facilities is presented in sections 4 and 5 respectively, and section 6 covers the interpretation of the test results. Between the commissioning of CAPTIF in 1986 and 2000, the SLAVE units have applied 8,400,000 load repetitions over 13 different trials. ALF was commissioned in 1984, and between 1984 and 2000, 25,000,000 load repetitions had been applied to over 200 sections of pavement in 25 different trials. Both facilities have addressed the following areas in various tests: traffic loading, asphalt and binders, unbound, recycled and stabilised road making materials, and pavement design and structural behaviour. In addition, ALF has also conducted research in road surfacings and markings, construction, maintenance and rehabilitation methods and asset management.

In general, accelerated pavement testing (APT) provides a useful link between laboratory testing on small samples and in-service pavements. APT is better than small scale testing as the loading and pavement structure are more realistic, and APT is better than in-service testing as the load and climate are either controlled or measured. However, APT cannot easily or quickly replicate seasonal changes in the moisture content of the pavement, nor can it accelerate the ageing of the bituminous materials. The scope of APT is usually limited by time and budget constraints, you can only test a very limited number of combinations of pavement materials, subgrade conditions, layer thicknesses, climatic conditions and loading conditions.

9.3 Cross Test

A test pavement with three different sections of high quality basecourse material was constructed. Two of the sections were constructed with high quality local aggregate, and the third section was constructed with high quality, quarried crushed rock imported from Australia. There was a marked difference in the performance of the three different pavement sections. The Australian material had a lower rate of initial rutting: however, after 50 kcycle, the rate of increase in rutting was the same for all test sections.

Deflections in the Australian material were also lower than the other sections. This was probably due to the subgrade being slightly stronger in this section of the tank as the deflections in this material remained lower than the other materials, even after the rate of rutting equalised.

The measurement of soil strains to a depth of 400 mm below the subgrade surface was successfully conducted for the first time, although the overall performance of the Bison system was disappointing with the reliability of the coils and/or cables being a major problem. It is thought that the high amounts of deformation affected the buried hardware. The replacement soil strain system from Nottingham University performed with 100% coil reliability after 600k load applications in the current project (C13).

Overall there were no problems with the importation and handling of the Australian material and the performance of the material was on a par with the TNZ specified material. The Australian material was more angular in shape than the local material. This was most likely due to the fact that this material was quarried from solid rock, unlike the local material, which was an alluvial aggregate.

Due to the (mainly moisture-related) difficulties encountered with the benchmark test at the ALF site, it was difficult to draw detailed comparisons between the performance of the two facilities. When the rate and amount of rutting was compared between the two facilities, the ALF pavement did not exhibit any signs of reduction in the rate of rutting, whereas the rutting in the CAPTIF pavement stabilised early in the test. This can be attributed to either the ALF pavement being loaded beyond its capacity in terms of load and/or high levels of moisture destabilising the pavement. The high FWD deflections (2 mm) on the ALF pavement suggest that the pavement could be considered weak. Although the CAPTIF deflections were similar, they were obtained at a higher contact pressure (700 kPa v 500 kPa) and load (40 kN v 35 kN).

The performance of the granular material, when tested by ALF and CAPTIF, was similar, which suggests that the results of testing at either facility can confidently be translated provided that other factors can be accounted for, particularly environment, i.e. height of water-table, temperature, subgrade conditions, etc. This implies that material need not be shipped between New Zealand and Australia provided compatible materials can be located. The controlled environment at CAPTIF makes it suitable for tests that require high numbers of load applications, because the deterioration of the pavement can be related to the number of load applications only (all other points being equal). Because the CAPTIF pavement is constructed under cover and contained in a tank, there is virtually no risk of uncontrolled water entering the pavement structure.

The results of this test reinforce the idea that CAPTIF is best suited to tests that involve either moisture-sensitive materials, or consistency in conditions between consecutive tests. The advantage of ALF is its ability to test either as-built pavements or pavements in the natural environment.

The recent commissioning of a shed in which ALF can be located will assist in the control of the environment, particularly if external drainage is controlled. As ALF pavements are not constructed within a concrete tank, variations in the height of the water-table could still pose some problems. To date, however, problems at the site where the benchmark test was conducted – and where the shed has been constructed – have been related to water entry through the surface, or from the side, rather than to variations in the height of the water-table.

10. Recommendations

Two sets of recommendations are put forward in this report. The first set covers the coordination of activities between the two facilities, and the second set covers areas or points that should be considered in future CAPTIF trials.

10.1 Proposal for the Coordination of Activities

The development and utilisation of accelerated pavement testing facilities such as ALF and CAPTIF is based on the premise that the research will eventually lead to savings in road agency and/or road user costs, which will arise principally in the following areas:

- initial investment costs of roads,
- maintenance of roads, and
- serviceability of roads.

Cooperation using the ALF and CAPTIF facilities for investigating pavement behaviour is desirable because of the diversity of the factors to be taken into account in pavement engineering, and the need to understand the mechanism of pavement deterioration. Coordination is also desirable if an attempt is to be made to reduce the number of different approaches to this problem and to optimise the use of limited resources.

Future coordinated activities could be structured in accordance with the directions outlined below, starting with the task of developing uniform rules for the application of results and ending with the in-depth exchange of detailed research information as follows:

- establish a procedure for the systematic exchange of information;
- investigate common points of future study programmes;
- set conditions for specific joint tests and studies on comparability of testing methods; and
- explore possibilities of cooperation in terms of the interpretation of results.

The establishment of an information exchange procedure requires the creation of a Working Group. The Group should be part of an official organisation such as Austroads and be managed through the Austroads Pavement Reference Group (APRG). Membership of the Working Group would include the ALF and CAPTIF Programme Managers and key members of each team, particularly operations staff, plus selected members of Austroads Member Authorities and industry.

10.1.1 Systematic Exchange of Information

Owing to time and resource constraints, daily tasks usually take precedence over the preparation and distribution of information. Information exchange would be more easily accomplished through a Working Group, which could meet once or twice a year, during which issues pertaining to the use of the facilities could be informally reviewed, and emerging issues identified and discussed.

These meetings could be supported by written information regarding the status of current work and proposals for future work. Generally, the staff associated with each research facility would have already prepared these reports for the client, and any additional work would be minimal.

A list of exchange themes about which information could be disseminated at least once each year – perhaps immediately after the yearly budget and programme is defined – is suggested in table 10.1. This could take the form of a written report on two aspects:

- the past year's activities; and
- the programme for the coming year.

The disseminating process should be revised on the basis of experience gained from actual and reciprocal information exchange, to ensure it functions well.

Currently the National Highway Co-operative Research Programme (NCHRP) in the United States is undertaking a project (NCHRP 10-56) which is attempting to develop a standardised set of terms and definitions for APT. The development of a common set of definitions will aid in the transfer of information and technology amongst all APT facilities around the world. Staff from the ALF and CAPTIF facilities are contributing to this project.

10.1.1.1 *Exchange Themes*

Exchange themes should be limited to the specific reasons for the test, conditions of execution of the test, description of the results, and application to real roads. A list of exchange themes associated with full-scale test facilities is suggested in table 10.2. An annual exchange of information is appropriate in view of the effective duration of the experiments. The speed of exchange could be increased by limiting this exchange of information between Australia and New Zealand, and limiting the information exchanged to completed, current or projected experiments.

Exchange of the various research reports on tests carried out on the two facilities is a minimum requirement. However, it would also be useful to exchange internal or interim reports describing any difficulties encountered, particularly, for example, with the use of instrumentation. This naturally infers some rules governing the confidential nature of this information.

Finally, both the agencies operating their respective accelerated testing facilities are well aware of the fact that each facility will always have to be considered as a simulation – varying in its reliability – of the real conditions of a road under normal traffic. Thus, it is necessary to define the "transfer function" between the behaviour of the experimental pavement and the known (observed and measured) behaviour of the pavements in the road network if readily useable results are to emerge from any tests. Such a complicated problem of interpretation of test results demands joint work, going beyond the exchange of information only.

Table 10.1 Proposed Rules for Systematic Annual Exchanges

Theme	Topic	Form of Presentation	Frequency (number of times/year)
1. Annual programmes	Programme for New Year (Forecast): ❑ Pavement structure ❑ Yearly budget ❑ Fund allocation Programme for past year (results)	Systematic Information: loading conditions versus time (forecast) Scheme (with figures) table summing loading conditions	1 (beginning of the year) 1 (end of the year)
2. Measurements conducted on the facility	Parameters which are measured ❑ Testing methods ❑ Functioning of sensors ❑ Evaluation of measurements ❑ Interpretation of results (performance monitoring)	1 or 2 pages on each	
3. Interpretation and use of results for in-service roads	Reports Interim reports Papers Articles	Depending on nature of publications Systematic information	End of each experiment Systematically for external publications

10.1.2 Cooperative Work Proposed for 2000/2001

It was recognised by APRG and the Austroads Asset Management Reference Group (AMRG) that an opportunity existed to improve the reliability of the link between traffic loading and pavement performance. And that rather than adopt a reactive mode, it would be more appropriate for Austroads to assist in the development of models which could be reliably used to assess the potential impact of new vehicle configurations on the pavement asset over a range of typical operating conditions.

If such models could be developed, then both Austroads and industry – including the heavy vehicle industry – would be better placed to respond positively to any emerging issues with respect to the potential impacts of future generations of heavy vehicle on pavement damage. Especially with respect to unbound granular pavements with thin bituminous surfacings, which comprise 95% of Australasia's sealed road network.

There is also a growing mass of evidence that the predicted performance of pavements incorporating thin surfacings, designed using the current method, is not being realised in the field. As in all pavement failures there is probably no single element that causes the observed lack of performance but a random combination of a number of elements.

Table 10.2 Suggested exchange themes between ALF and CAPTIF

1. Description of the facility

1.1 Precise description of the equipment
1.2 Operating procedure
1.3 Theoretical number of load passes and costs
1.4 Operating efficiency, utilisation rate
1.5 Explanation of the gap
1.6 Reliability, breakdowns, real costs
1.7 Improvements (reasons)
1.8 Realism of simulation

2. Studies of structural design of pavements

Before trial:
2.1 Pavement structures for trials - place in a general research programme
2.2 Prediction of the theoretical performance for the track
2.3 Construction conditions
2.4 Material qualities, mechanical behaviour
2.5 Internal equipment of the test section
2.6 Differences between the real construction and the projected one
2.7 New prediction of the performance

During trial:
2.8 Proposed test schedules
2.9 Pavement response:
- vertical deflection
- strain gauges
- other sensors
- various testing methods

2.10 Environmental factors
2.11 Evolution during progress
2.12 Link with theoretical modelling of pavements

3. Pavement performance monitoring

3.1 Failure criteria
surface of road
- profile measurement
- roughness
- wear indicator

3.2 bearing capacity
- permanent deformation
- cracking indicator

3.3 Variation of chosen indications during testing
3.4 Combination of the various indicators with respect to failure criterion

4. Calibration and application of results for road in service

4.1 Interpretation of the indicators for a failure criterion
4.2 Extrapolation of results
4.3 Verification of the adopted procedure

In addition, Transit New Zealand is currently undertaking an extensive review of mass limits and introduction of new vehicle types into that country. Transfund are funding a project examining the performance of granular pavements under 8 tonne and 10 tonne axle loads and proposing further work – in cooperation with Austroads – in 2001/2002.

During 1999/2000, Austroads agreed to support a programme of work that has, as its main objective, the development of a more rational basis for assessing the potential impacts of future vehicle configurations on the road pavement asset over a range of typical operating conditions. Especially with respect to unbound granular pavements with thin bituminous surfacings, which comprise 95% of Australasia's sealed road network.

During 1999/2000, literature reviews were conducted on axle loads which caused equivalent damage, and the relevant damaging effects of wide single and dual tyres (Foley 2000*a* and *b*). In addition, a consultant was commissioned to survey industry with respect to likely future vehicle configurations (Foley and Pearson 2000).

The project is being supported by AMRG who wish to ensure that current pavement performance models adequately predict the consequences of increased axle mass loadings on pavement condition. The current pavement performance models are not likely to be adequate for this purpose because loadings imposed in this manner have not been experienced before and the current theoretical bases are therefore not adequate.

The objectives of the project in 2000/2001 are to:

- build on the information collected during 1999/2000 and identify the issues that need to be addressed with respect to the performance of unbound granular pavements with thin bituminous surfacings which are likely to be affected by existing and emerging vehicle configurations – and to determine how these issues can best be addressed;
- conduct a pilot cooperative Austroads/Transfund accelerated pavement testing trial – with emphasis on response to load investigations – to investigate some of these issues; and
- develop a proposal for further studies to provide input into the development of a framework to assess the potential impacts of future vehicle configurations on the road pavement asset over a range of typical operating conditions.

It is envisaged that, from 2001/2002 testing will be conducted – using both CAPTIF and ALF – which can address some of these issues.

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APPENDIX A

TRANSVERSE PROFILE HISTORIES: CAPTIF CROSS-TEST

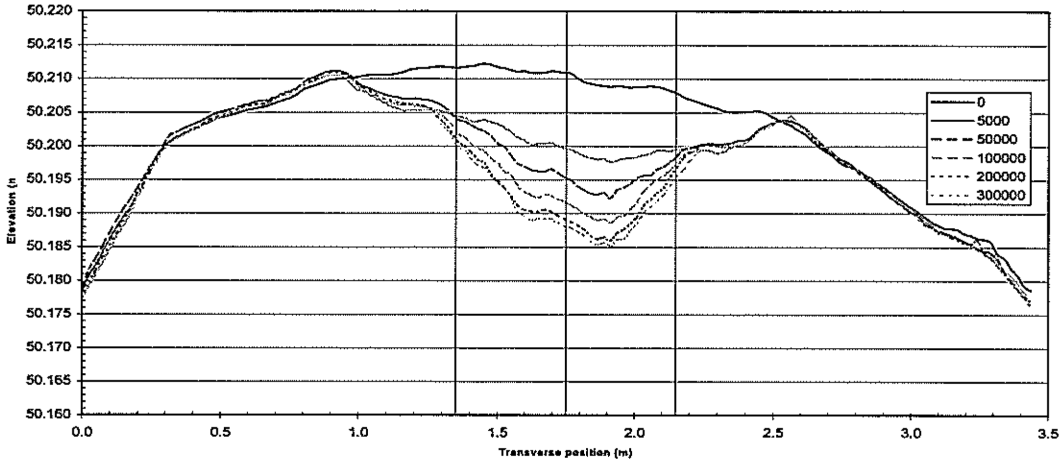

Figure A1: Transverse profile history for Station 4

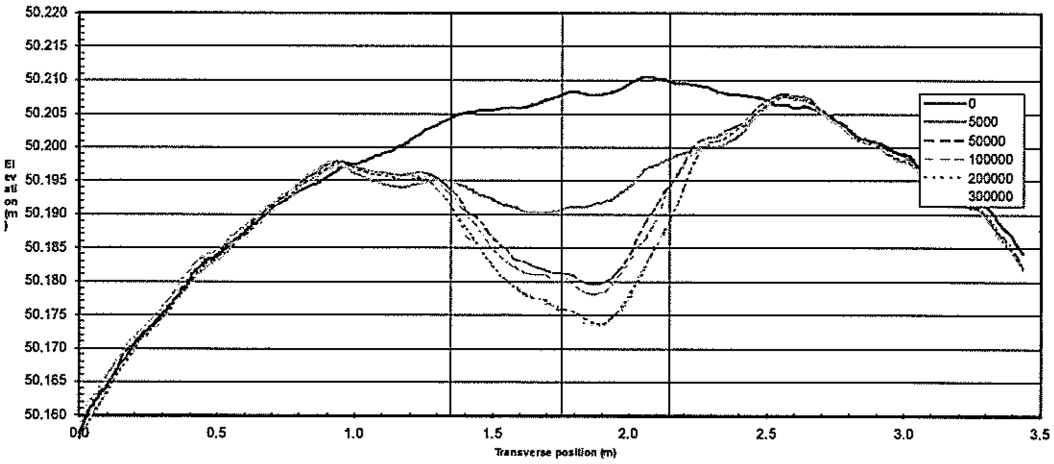

Figure A2: Transverse profile history for Station 14

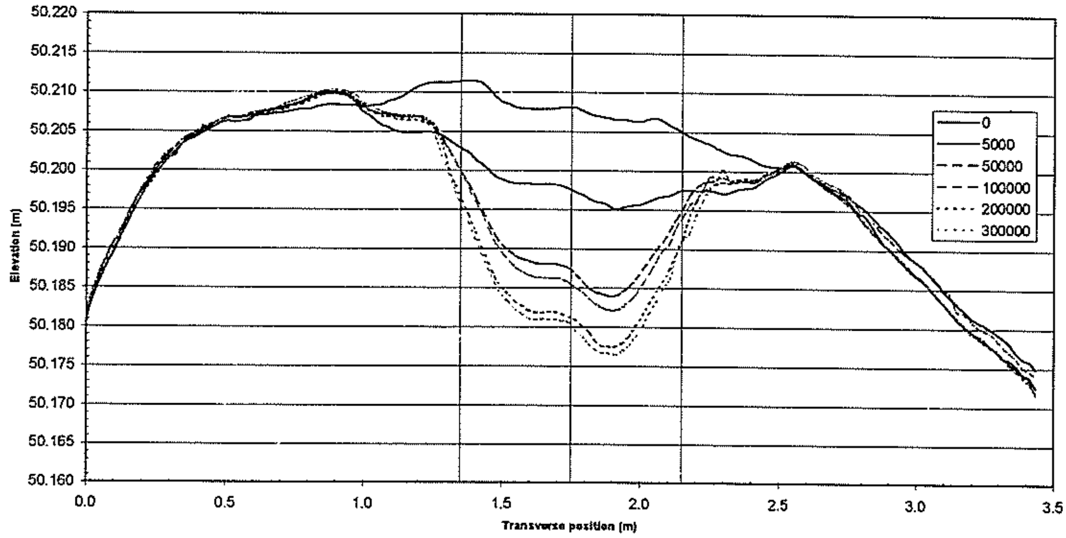

Figure A3: Transverse profile history for Station 24

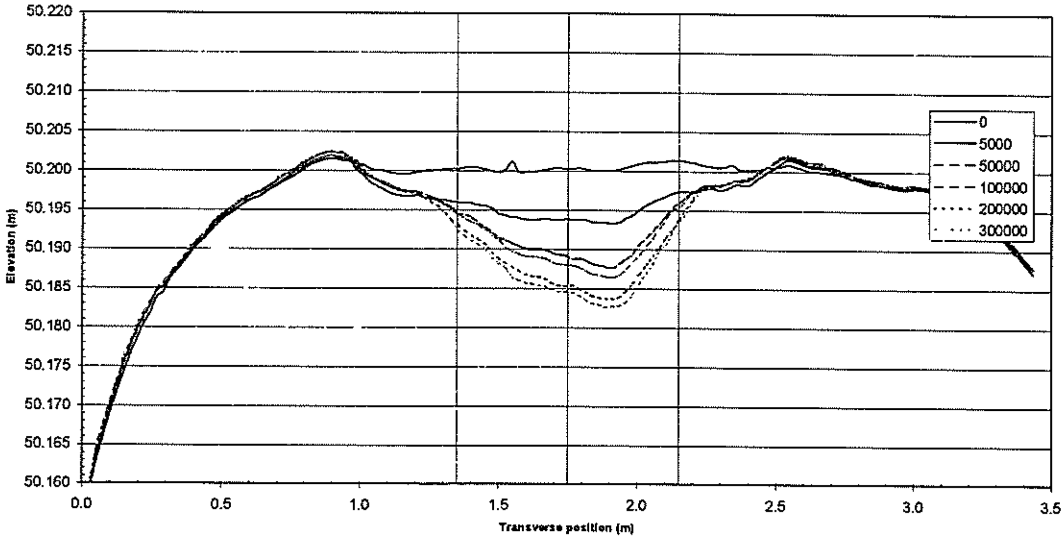

Figure A4: Transverse profile history for Station 31

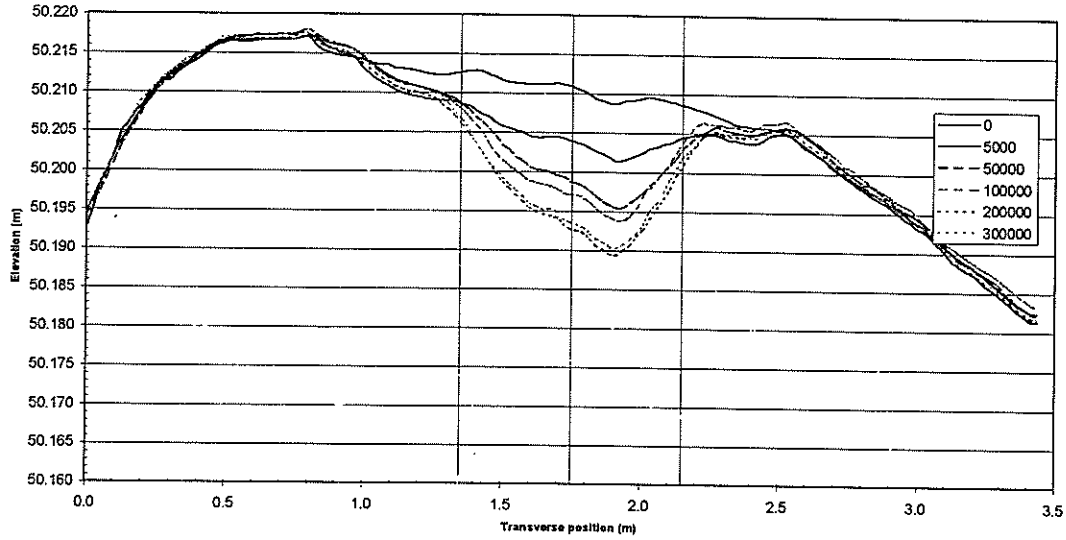

Figure A5: Transverse profile history for Station 51

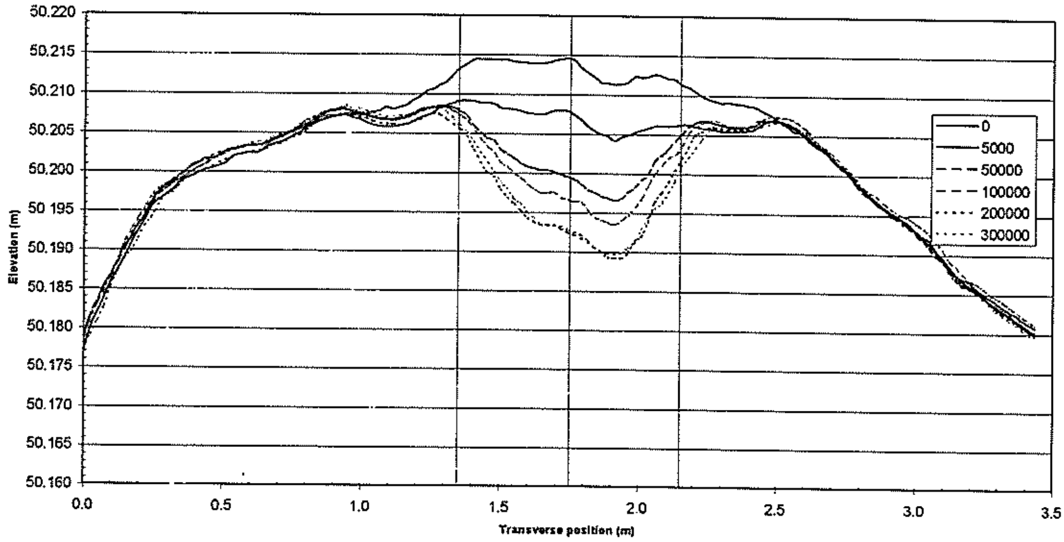

Figure A6: Transverse profile history for Station 54

APPENDIX B

INITIAL AND FINAL LAYER INTERFACES: CAPTIF CROSS-TEST

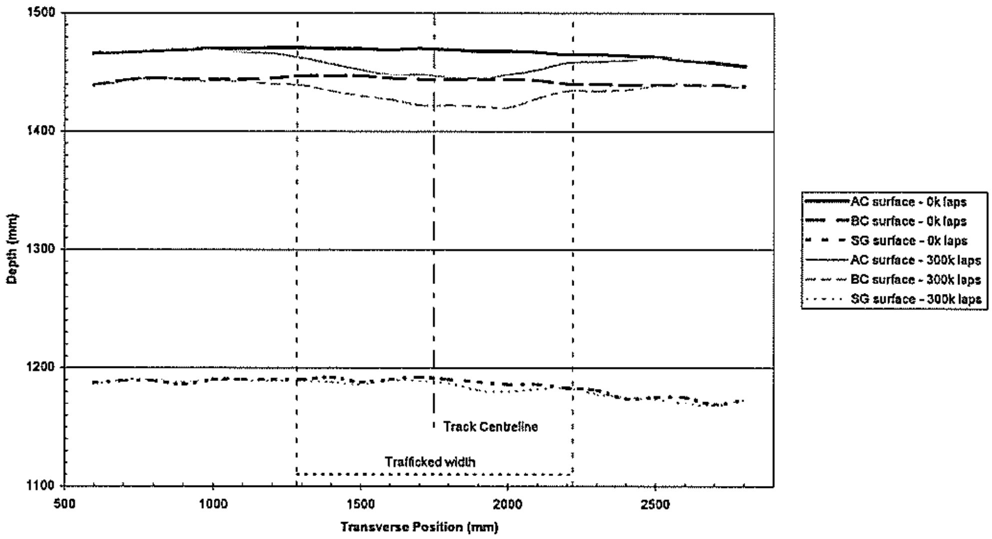

Figure B1: Initial and final layer interfaces for Station 4

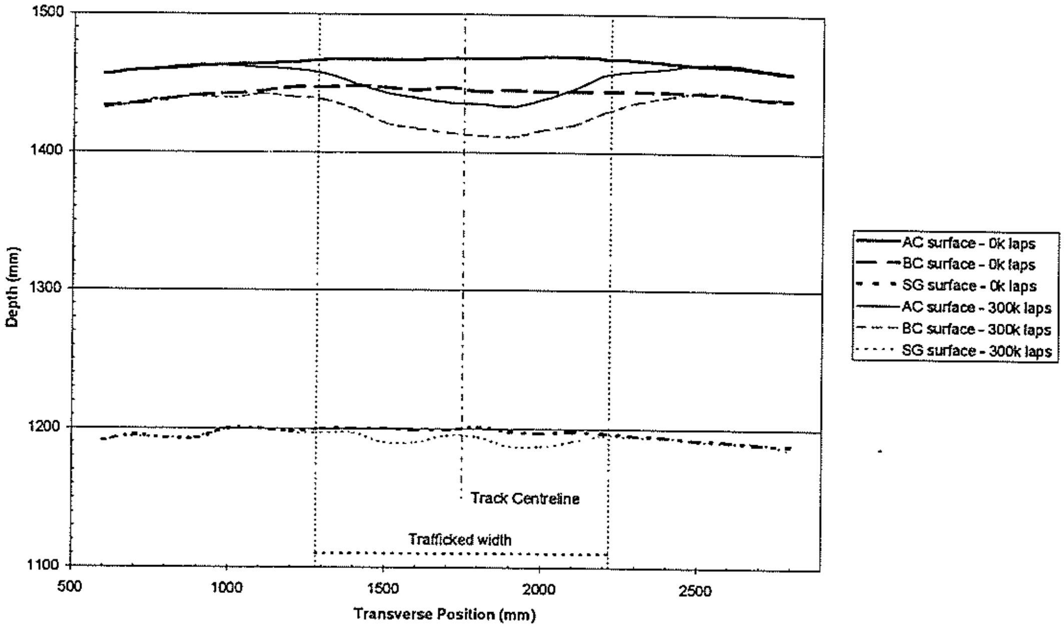

Figure B2: Initial and final layer interfaces for Station 14

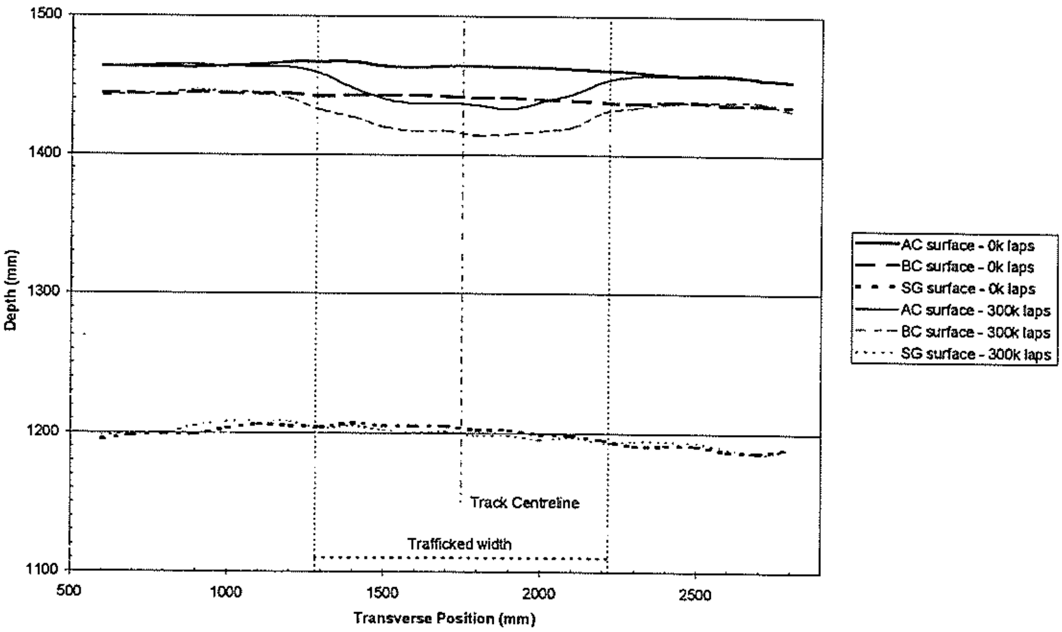

Figure B3: Initial and final layer interfaces for Station 24

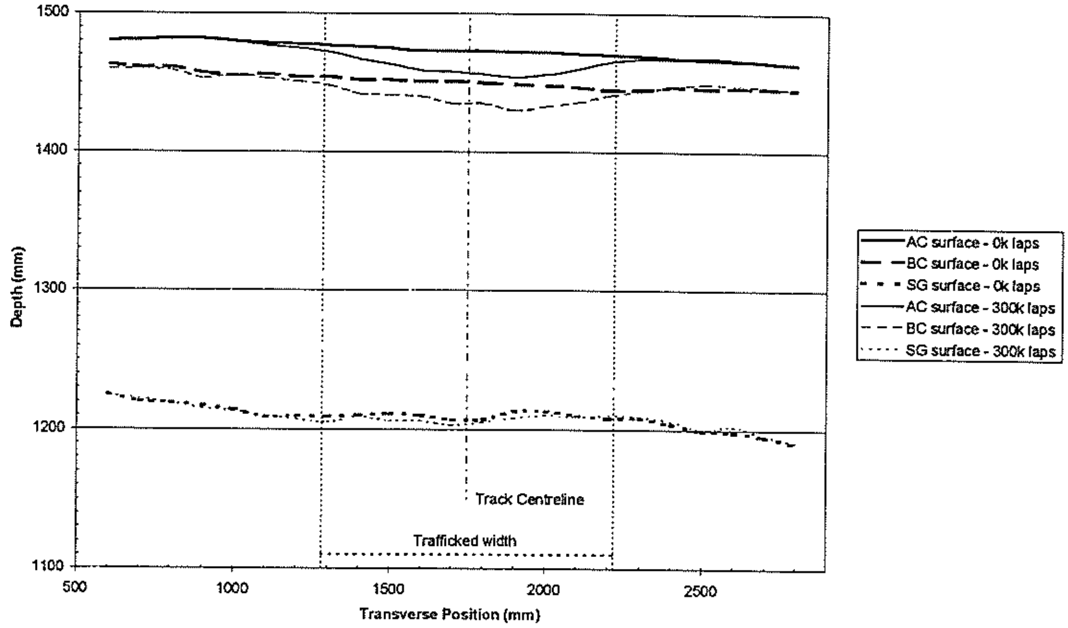

Figure B4: Initial and final layer interfaces for Station 31

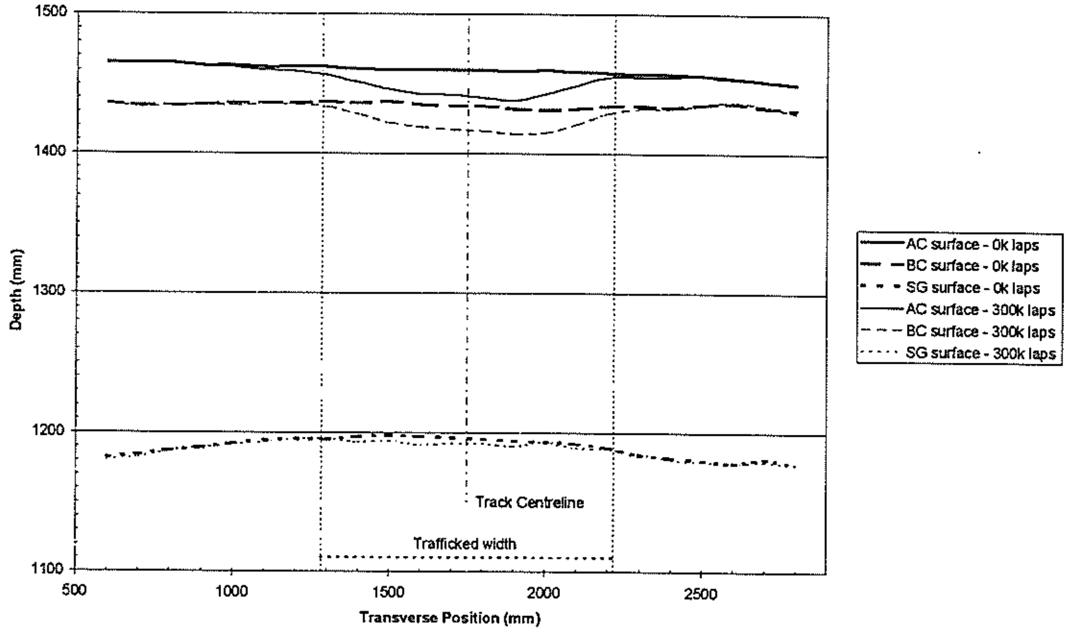

Figure B5: Initial and final layer interfaces for Station 51

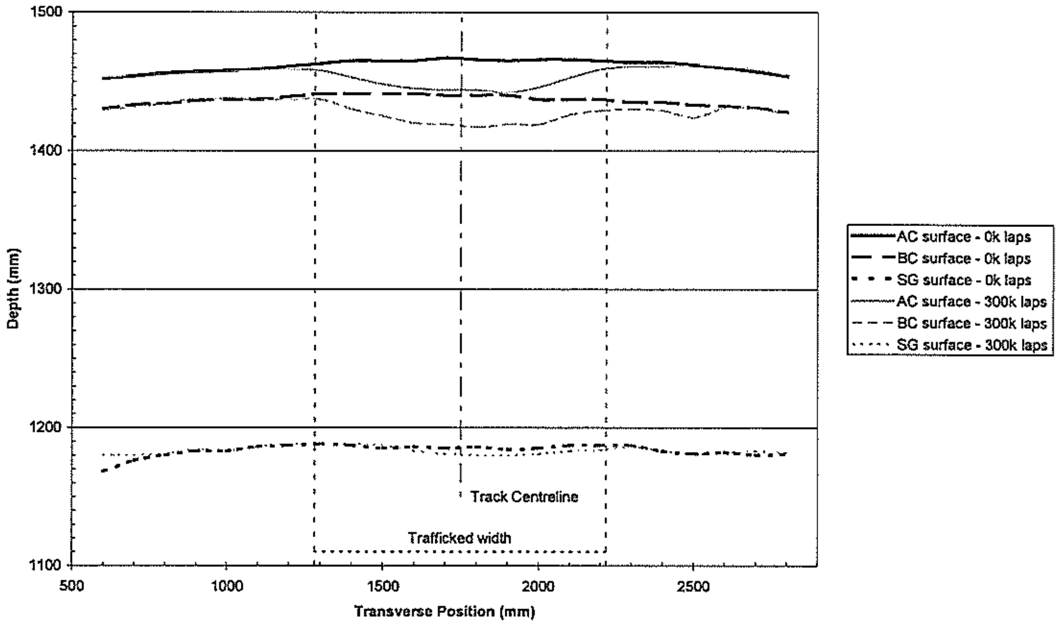

Figure B6: Initial and final layer interfaces for Station 54

Figure 3.3 View of Falling Weight Deflectometer

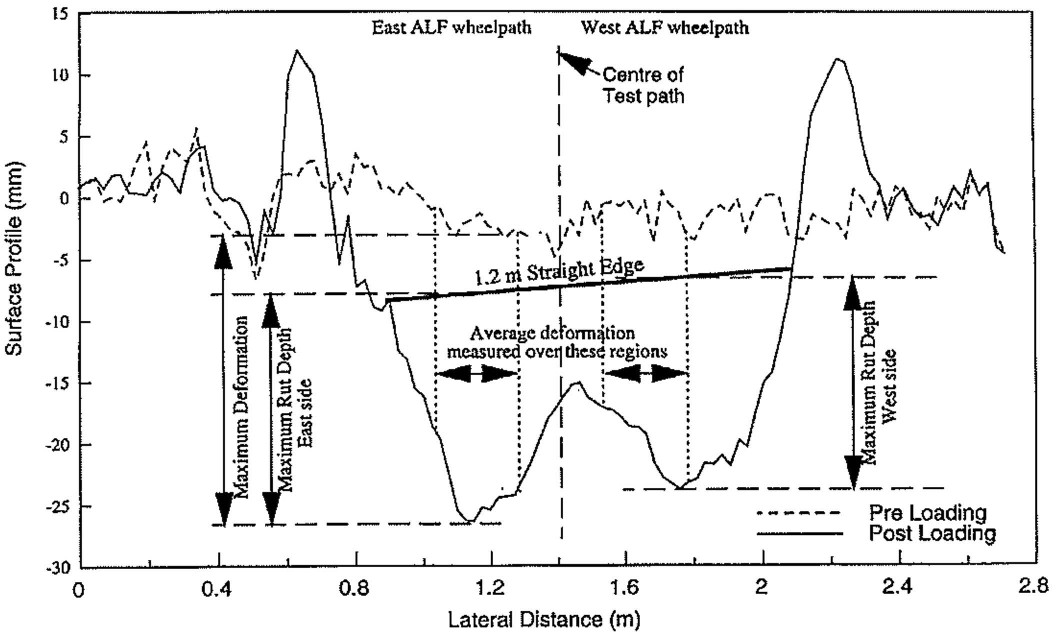

Figure 3.4 Typical transverse profilometer data

3.3.1 Data Acquisition and Processing System

The response of the test sections to the passing ALF wheel load (deflections) and permanent deformation is measured at regular intervals by displacement transducers connected to ARRB TR's Digital and Analogue Data Acquisition System (DADAS). The outputs of these devices, in the range of -1 to +1 V, are converted to digital units ranging from -2048 to +2047 and stored in solid-state memory of the DADAS from which they may be retrieved and stored on the hard disk of a Personal Computer. This allows both the quality of data to be controlled, and problems with the instrumentation and test equipment to be detected.

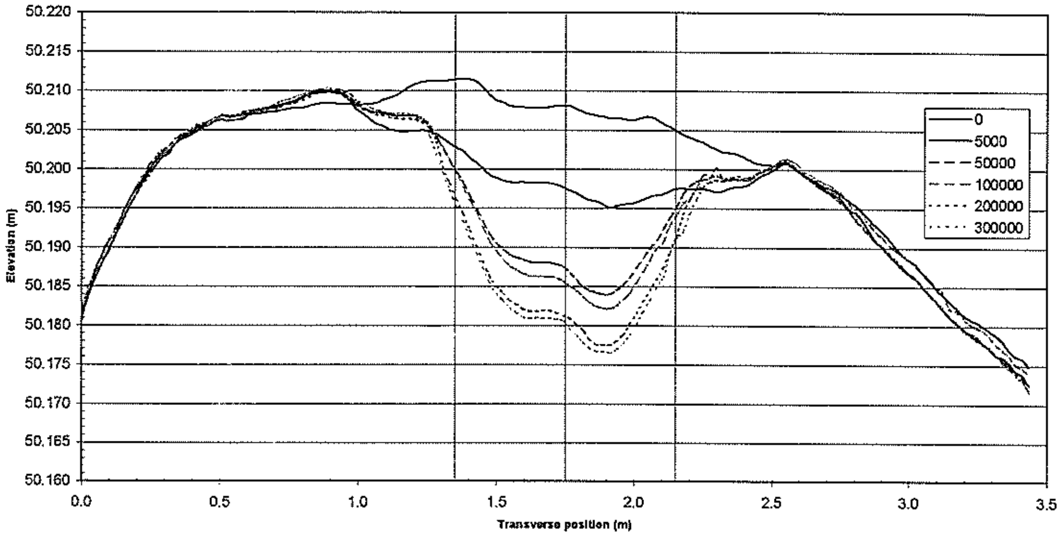

Figure A3: Transverse profile history for Station 24

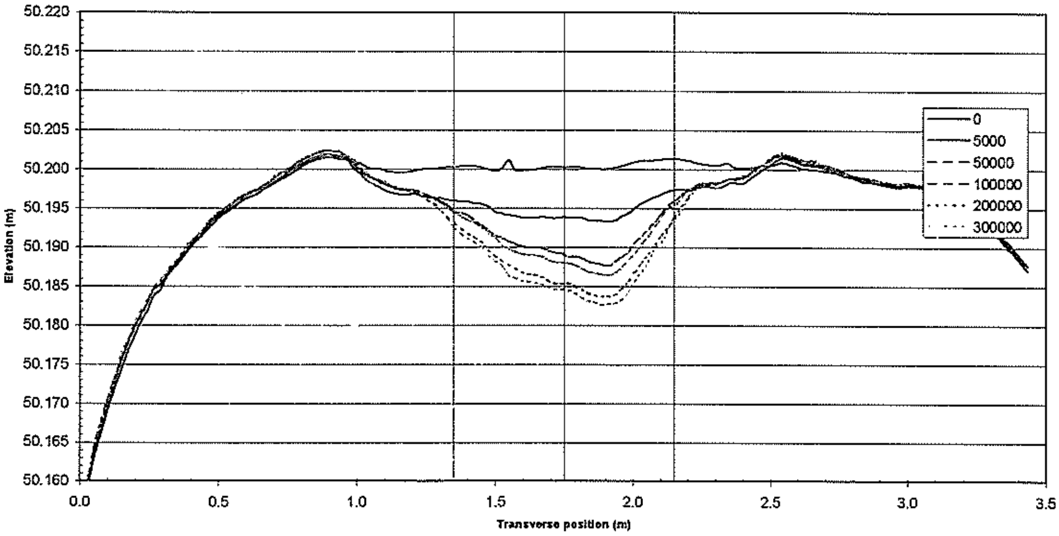

Figure A4: Transverse profile history for Station 31

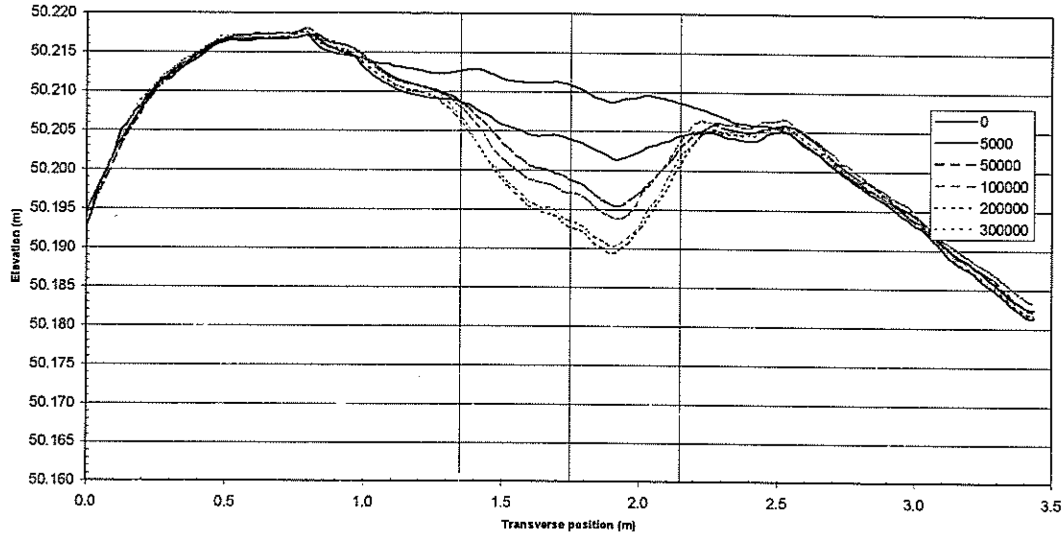

Figure A5: Transverse profile history for Station 51

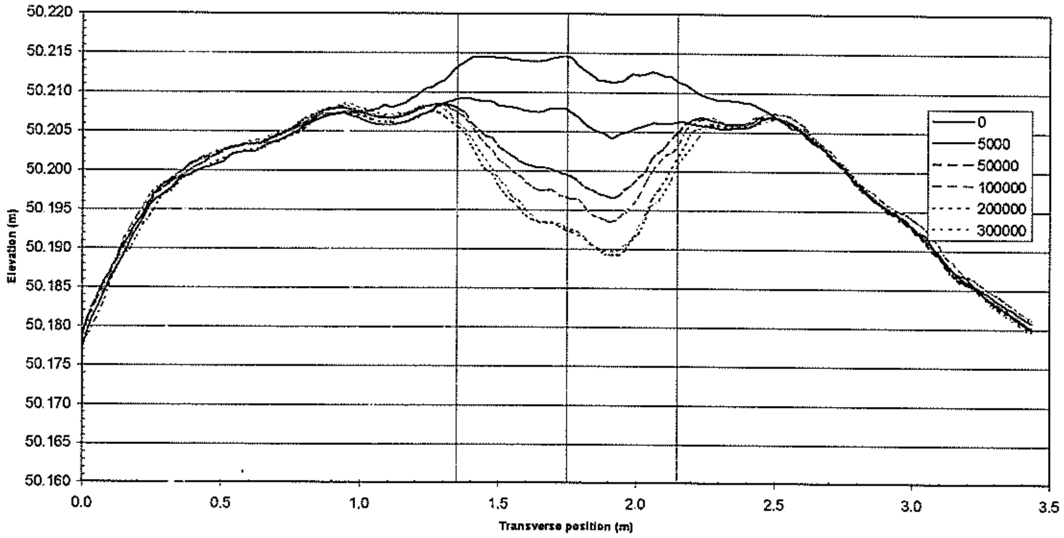

Figure A6: Transverse profile history for Station 54

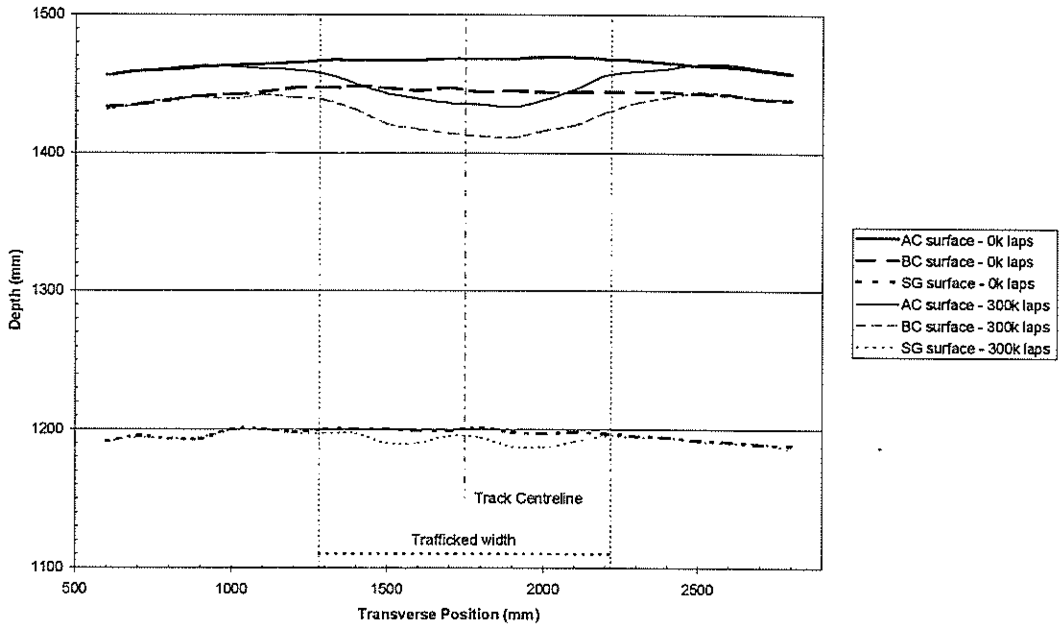

Figure B2: Initial and final layer interfaces for Station 14

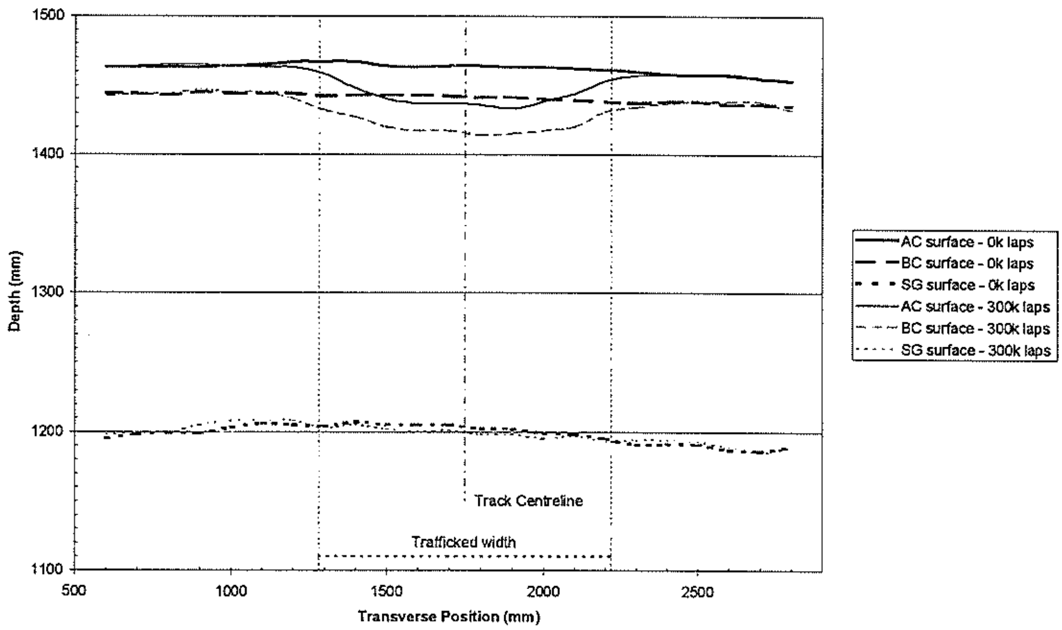

Figure B3: Initial and final layer interfaces for Station 24

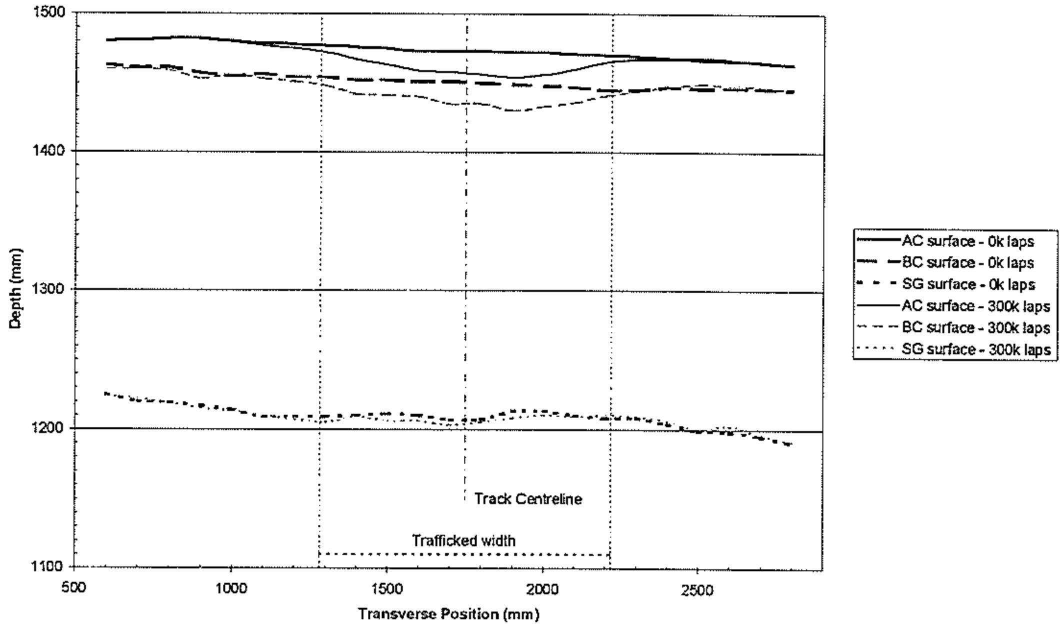

Figure B4: Initial and final layer interfaces for Station 31

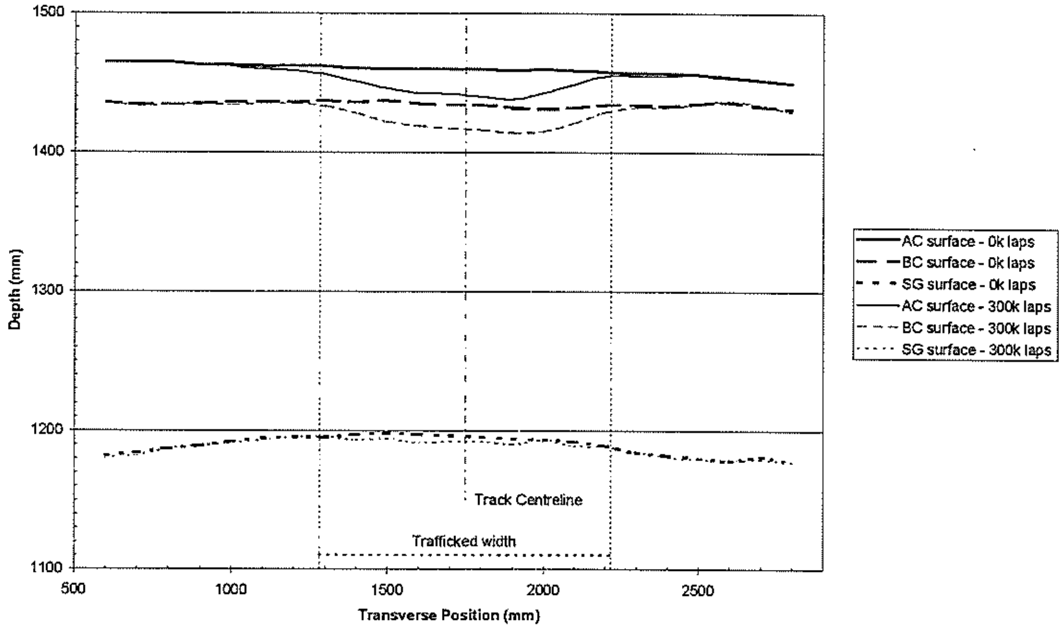

Figure B5: Initial and final layer interfaces for Station 51