THE RELATIONSHIP BETWEEN DYNAMIC WHEEL LOADS AND ROAD WEAR

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AN IMPORTANT NOTE FOR THE READER

The research detailed in this report was commissioned by Transfund New Zealand.

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GLOSSARY

AASHO	American Association of State Highway Officials *(before December 1973)*
AASHTO	American Association of State Highway and Transportation Officials *(after January 1974)*
AC	Asphaltic Concrete
ARRB	Australian Road Research Board
AUSTROADS	National Association of Road Transport & Traffic Authorities in Australia
CAPTIF	Canterbury Accelerated Pavement Testing Indoor Facility
CBR	California Bearing Ratio
DIVINE	Dynamic Interaction of Vehicles and INfrastructure Experiment
DLC	Dynamic Load Coefficient
EC	European Commission
EDA	Equivalent Design Axle
ESA	Equivalent Standard Axle
FWD	Falling Weight Deflectometer
IRI	International Roughness Index
LCPC	Laboratoire Central des Pontes et Chaussées
LVDT	Linear Variable Differential Transformer
MATTA	MAterials TesTing Apparatus
NAASRA	National Association of Australian State Road Authorities
NRB	National Roads Board
OECD	Organisation for Economic Co-operation and Development
OECD RTRP	OECD Road Transport Research Programme
PGSF	Public Good Science Fund
psd	particle size distribution
PSD	Power Spectral Density
SLAVE	Simulated Loading And Vehicle Emulator
TNZ	Transit New Zealand
VMA	Voids in the Mineral Aggregate
WLPPM	Whole Life Pavement Performance Model

EXECUTIVE SUMMARY

Introduction

Conventional pavement design and management practice is based on the static axle loads of the vehicles that traffic the pavement. Since the AASHO road test of the late 1950s it has been assumed that the pavement wear generated by an axle is proportional to some power of its static axle load. The most commonly used value for this power is four.

Real axle loads are, of course, not static but dynamic and vary with suspension type and vehicle load as well as speed and road surface profile. Two vehicles with the same static axle loads can generate very different dynamic loads. It has been postulated that the use of suspensions that reduce the level of dynamic loading will generate less pavement wear for the same level of static load. This seems intuitively reasonable and has been shown theoretically using the fourth power relationship between load and wear described above, together with some assumptions regarding the load distribution.

If the hypothesis is correct these "road-friendly" suspensions offer an attractive opportunity to pavement managers. Pavements could carry the same traffic load for less wear or alternatively carry more traffic load for the same wear. Any policy initiatives to encourage greater use of road-friendly suspensions require an understanding of the relationship between dynamic wheel loads and pavement wear.

In New Zealand the situation is further complicated by the fact that most of our pavements are thin-surfaced unbound granular structures. The behaviour of these pavements is different from that of the asphaltic concrete (AC) flexible pavements extensively used in the European and North American road networks. Most of the international body of research on flexible pavements relates to these AC pavements (for example, the fourth power relationship between loads and wear), but the results have been applied to the New Zealand style thin-surfaced pavements anyway.

Project Overview

This research project consisted of a series of three accelerated pavement tests to determine the relationship between different levels of dynamic loading and pavement performance and life. The tests were undertaken at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) between 1993 and 1998. CAPTIF is unique among accelerated pavement testing facilities in that it is designed to generate realistic dynamic wheel loads. All other accelerated pavement testing facilities that we are aware of try to minimise dynamic loading. Originally the Simulated Loading And Vehicle Emulator (SLAVE) units, which are a "quarter truck" type of vehicle that applies the pavement loads at CAPTIF, were fitted with a multi-leaf two stage steel spring suspension. To undertake this project, and as part of the general improvements and development at CAPTIF, the SLAVE units were modified to be able to be fitted with two other suspension configurations.

For each of the tests the two SLAVE units were fitted with different suspension configurations but with the same static load, and operated in separate parallel wheel paths on the same pavement. The two different suspensions resulted in different levels of dynamic loading being applied to the pavement under the vehicles. By monitoring the performance and life of these two wheel paths we can determine the influence of dynamic loading.

First Pavement Test

The first test involved a New Zealand style thin-surfaced pavement structure with a design life, based on the National Roads Board (NRB) design charts, of 350,000 ESA (Equivalent Standard Axle). One of the SLAVE units was fitted with the original multi-leaf steel spring while the other was fitted with a modern steel parabolic leaf spring and a viscous damper. Wide-based single tyres were used to maximise the separation between the two wheel paths, and the static wheel load was 37kN which is approximately equal to half the reference weight for an axle fitted with wide single tyres (7.2 tonnes). Thus each load cycle would apply approximately 1 ESA to the pavement.

This pavement failed after only 35,000 load cycles. At this point the surface profile had deteriorated to the extent that it was not possible to continue operating the facility safely. Although the parabolic spring suspension, being a more modern design, had been expected to be more road-friendly than the multi-leaf steel spring, this was not the case. From the start the dynamic loads generated by the SLAVE with the parabolic spring were higher than those from the multi-leaf at the test speed. An analysis of the suspension characteristics showed that, although the parabolic spring had less hysteresis than the multi-leaf spring, it was stiffer. Similarly although it had viscous damping (a positive feature), the damping levels were too low to be effective. Part of the reason for these poor characteristics was that the suspension was designed for the SLAVE maximum wheel load of 60 kN and consequently was considerably over-rated for the actual load of 37 kN.

This situation has parallels with in-service vehicles. Many of the vehicles in the New Zealand road fleet are sourced from countries with higher allowable axle loads than New Zealand, and it is not uncommon for these vehicles to be fitted with suspensions that are rated for these higher loads. The wheel path trafficked by the parabolic spring suffered the most damage which was in accord with the pattern and magnitude of the dynamic loads. A more serious concern was that the pavement achieved only about 10% of its design life. Generally, pavements at CAPTIF last longer than the design life because of the protection from environmental factors there. The damage consisted of severe rutting in the basecourse layer with considerable lateral movement of material. The post-mortem analysis suggested that this was caused by inadequate compaction of the basecourse layer during construction. In spite of the premature failure of this pavement and consequently the small amount of data that was collected, it was possible to show a correlation between dynamic wheel forces and pavement profile and profile change. While this relationship had long been postulated this was the first time anywhere in the world that it had been measured.

Second Pavement Test

The second pavement test in the series was undertaken as part of the OECD DIVINE (Dynamic Interaction between Vehicles and INfrastructure Experiment). To meet the requirements of the OECD scientific expert group overseeing the experiment and the interests of all the OECD member countries, a thicker AC pavement design was specified. While the asphalt layer was relatively thin (85 mm) by European and North American highway standards, so that the failure conditions might be achieved with a reasonable time frame, it was thick enough for its behaviour to be considered representative of asphaltic concrete flexible pavements. The inclusion of a thicker pavement in this series of tests has the merit of providing a link between the performance of conventional AC-surfaced flexible pavements and the thin-surfaced unbound granular structures used in New Zealand.

For this test, one of the SLAVE units was fitted with the multi-leaf steel spring while the other was fitted with an airbag spring and viscous damper. The static wheel load was set to 49 kN and again wide-based single tyres were used. The higher wheel load is in line with European practice and was also chosen to accelerate the wear. A 10-m section of the pavement was intensively instrumented. The test proceeded for 1,700,000 load cycles when it was terminated for time and cost reasons. At this point the pavement had still not reached the pre-set failure criteria and, at the rate of wear that was occurring, could have lasted up to 3,000,000 load cycles or more. This compares with most modelling predictions for this pavement of a life of 500,000 load cycles or less.

The two different suspension systems provided significantly different levels of dynamic loading, with the steel suspension more than twice as high as the air. A localised rut formed in the wheel path being trafficked by the steel suspension early in the test but then stabilised. This rut appears to have been the result of a combination of high dynamic loading and a local weakness in the pavement. This pavement weakness was not detectable in the structural capacity measurements done at construction. Because of the complication of this rut many of the analyses were undertaken on two data sets – a complete set comprising data from the whole pavement, and a reduced set without data from the rutted section.

Without the data from the rutted section of pavement, the mean levels of both rutting and cracking were similar in both wheel paths. For the complete data set, both rutting and cracking were higher for the wheel path trafficked by the steel suspension. However, the variation in both rutting and cracking was significantly higher in the wheel path trafficked by the steel suspension even for the reduced data set. Correlation analysis showed moderate relationships between dynamic wheel forces and the surface profile changes (rutting, longitudinal profile changes and vertical surface deformation) for the steel suspension, and very weak relationships for the air suspension. Conversely the relationship between initial pavement strength and surface profile changes is much stronger for the air suspension than for the steel. This implies that the surface profile changes in the pavement are related to both the initial structural capacity and the level of dynamic loading. If the level of dynamic loading is kept low as with the air suspension then the surface profile changes are driven by the variations in structural capacity. If the pavement structure is also uniform, then the pattern of wear will be uniform as well. On the other hand with high dynamic loads, an initial local pavement weakness will lead to some local pavement deformation that will stimulate a dynamic response. This will result in a pattern of dynamic loading that will contribute to further surface profile changes. The net effect is that the wear is more localised with areas of more severe damage and other areas with less damage.

Third Pavement Test

The third pavement test was undertaken on a New Zealand-style thin-surfaced pavement design similar to the first test but with the same vehicle and loading configuration as the second test. Although the design life of this pavement was 94,000 load cycles, when the test was terminated at 300,000 load cycles the pre-set failure criteria had not been met. The key pavement wear factor for this pavement was permanent deformation. The mean level of wear was similar for the two wheel paths but the variation in wear was related to the level of dynamic loading. As with the previous pavement there was a moderate correlation between pavement wear and dynamic wheel loads, and also between wear and pavement structural capacity. The degree of correlation was approximately the same for both wheel paths.

Conclusions

- Overall it appears that pavements at CAPTIF last much longer than the design life predicted by the old NRB design charts and by many of the pavement life models, possibly up to three times or more as long. This phenomenon has also been observed on previous pavement tests. However, the AUSTROADS design guide, which was adopted by Transit New Zealand in 1995, predicts pavement load cycle limits that are much closer to those observed at CAPTIF.

  The main differences between CAPTIF pavements and in-service pavements appear to be the reduction in environmental influences at CAPTIF, the relatively high degree of quality control of materials and construction, and the accelerated testing, which reduces aging effects. If these are the reasons for an increased pavement life at CAPTIF then it implies that some pavement wear can be attributed to environmental, construction quality, and aging effects rather than to loading. This has major implications for road pricing and user charges policies.

- The mean level of pavement wear observed for the last two tests appears to be independent of dynamic loading. While this might be interpreted as indicating a first power relationship between load and wear rather than a fourth power, this is not necessarily the case. The difference in mean wear that would be expected by applying a fourth power to the different dynamic loads is relatively small. Given the accuracy of the measurements, the possibility of a fourth power relationship cannot be conclusively eliminated. Lower powers (between one and four) could also fit the data. The AUSTROADS load cycles versus subgrade strain relationship uses a power of 7.14. While the pavement life values predicted by this relationship seem to have some validity, it is difficult to see any evidence for the power value being this high.

- The variations in pavement wear around the track are related both to the dynamic loading and to the variations in structural capacity of the pavement. These relationships, while expected, have never been measured anywhere before. Lower dynamic loads would result in a more even distribution of pavement wear. If the maintenance intervention criteria are set as a level of roughness or a maximum rut depth, this will increase the interval between interventions. Improving the uniformity of pavement structures by better construction practices and quality assurance should similarly reduce the frequency of maintenance required.

- The mechanisms of pavement damage observed were different to those predicted in the design models. In particular, compression of the basecourse layer was a significant contributor to the permanent deformation. With the second pavement, most of the cracking in the AC layer initiated from the top of the layer rather than from the bottom as predicted by most models. This phenomenon has been reported elsewhere and needs further investigation.

- Objective one of the PGSF vehicle-road interaction programme is developing a Whole Life Pavement Performance Model (WLPPM) using data from these tests and from the OECD DIVINE for validation. This model includes the development of sub-models for fleet dynamic loading effects, pavement response to loading, and pavement damage. These three sub-models address the main research needs identified by this project. The validation and implementation of this model will be enhanced through good quality pavement performance data from in-service pavements.

Recommendations

- Improved quality control and higher standards of uniformity and consistency during pavement construction should be promoted, as these will improve pavement performance. Some further investigation to quantify the benefits and costs may be needed.

- The relative magnitude of pavement wear costs associated with traffic loading compared with those related to environmental factors should be determined. This is an important factor in allocating costs to users.

- The relationship between vehicle loads and wear for thin-surfaced pavements needs further investigation. Although the results of this research do not eliminate the fourth power relationship, which is currently widely used, they do not provide strong support for it either.

- Road-friendly suspensions generating lower dynamic wheel loads should be encouraged as their use will reduce pavement maintenance requirements.

- Further research work is needed to quantify this effect and in particular the vehicle fleet effects. Work in this area is in progress through the Public Good Science Fund (PGSF) vehicle–road interaction programme.

- The mechanisms by which vertical surface deformation occurs on in-service pavements, particularly on thin-surfaced structures, need to be resolved. This is fundamental to the design and management of the New Zealand roading network.

- Crack initiation at the top of the AC layer has been reported elsewhere in the world and needs further investigation. However, in the context of New Zealand's predominantly thin-surfaced pavements, this is not a high priority.

- There is a need to investigate the reasons for the discrepancy between the measured and calculated strain values in the pavement. This stress-strain relationship forms the basis of the pavement design model.

- A long-term pavement performance monitoring programme should be established in New Zealand to provide good quality information on the performance of the roading network.

ABSTRACT

The models of traffic-induced pavement wear, which form the basis of current pavement design and management practices, are based on static axle loads. Real vehicles, however, are dynamic systems and generate dynamic loads, which depend on the vehicle characteristics (mass, load distribution, suspension stiffness and damping, wheelbase, tyre pressures), its speed, and the geometry of the pavement.

This report describes a series of three accelerated pavement tests undertaken, between 1993 and 1998, at the Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF) in Christchurch, New Zealand, to determine the relationship between dynamic wheel loads and pavement wear, particularly for the thin-surfaced unbound granular pavement structures widely used in New Zealand.

The first test used a New Zealand-style thin-surfaced pavement structure with a nominal design life of 350,000 load cycles trafficked by one SLAVE (Simulated Loading And Vehicle Emulator) unit fitted with a multi-leaf steel spring without viscous damping, and another fitted with a more modern parabolic leaf spring with viscous damping. The pavement failed prematurely after only 35,000 load cycles.

The second test was a key part of the OECD DIVINE (Dynamic Interaction of Vehicles and INfrastructure Experiment) project, comparing the effects of loads generated by an air spring suspension with viscous damping with those from a multi-leaf steel spring. An asphaltic concrete surfaced pavement, with an expected design life of about 500,000 load cycles or less, withstood 1,700,000 load cycles without reaching the failure conditions.

The third pavement test used the same vehicle loading configurations as the second test but on a thin-surfaced pavement structure similar to the first test. This pavement had a design life of 94,000 load cycles, and lasted 300,000 load cycles without reaching failure conditions. This test provides a useful cross-link between New Zealand-style pavement designs and thicker asphalt concrete pavements used in Europe and North America, on which much of the international research is based.

Conclusions are that the wheel path subjected to higher dynamic loading showed a significantly greater variation in wear than the wheel path with lower dynamic loads. Reducing the structural variability in the pavement structure and reducing dynamic loading will result in a more uniform distribution of wear and will lower the maintenance requirements. Both pavements withstood considerably more load applications than predicted by most design models, though the AUSTROADS pavement design guide provides the most accurate predictions of pavement life. If the design models accurately reflect the performance of in-service pavements, construction variability, aging and environmental influences are suggested as significant contributors to pavement wear.

1. INTRODUCTION

1.1 Overview

The AASHO road test of the late 1950s led to the development of the fourth power law, which states that the amount of pavement wear generated by the passage of an axle is proportional to the fourth power of the static axle load. This is a heuristic rule based on curve fitting to the observed data. Consequently it implicitly includes dynamic loading, environmental factors and anything else that contributed to pavement wear during the tests. The assumption was that all these factors were identical for the different pavements and loads involved. It should be noted that the overwhelming majority of pavements in the New Zealand road network consist of thin surfaced flexible structures, which were virtually ignored in the AASHO test. Consequently the validity of using the fourth power law in this context is questionable.

Experimental research programmes in the 1970s and 1980s (Sweatman 1983, Hahn 1985, Woodrooffe et al. 1986, Mitchell & Gyenes 1989) have measured the dynamic wheel forces generated by heavy vehicles and compared the performance of various suspensions. These studies have all shown significant differences in the level of dynamic wheel force generated between different suspensions under the same test conditions.

Although there is considerable debate over the validity of the fourth power law and, in particular, whether four is the correct value for all forms of pavement design and distress, it is widely accepted that some form of power relationship is appropriate for relating pavement wear to applied axle loads. With any power value greater than unity, higher dynamic loading would be expected to lead to increased pavement wear.

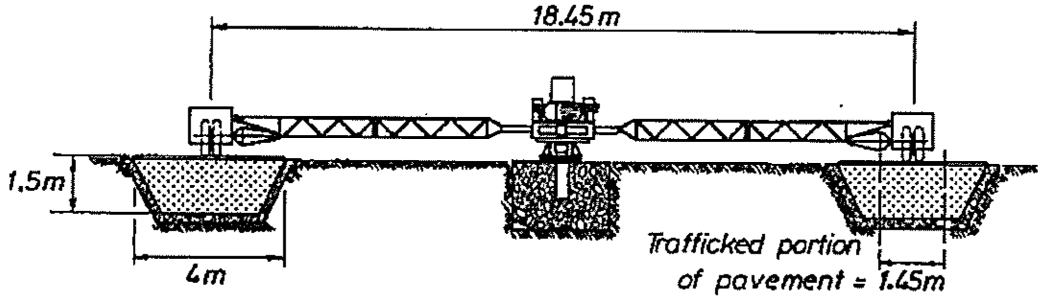
In this research programme, carried out between 1993 and 1998, a series of three accelerated loading pavement tests were conducted, where the effects of different levels of dynamic loading (but identical static loading) on pavement wear were compared.

1.2 Canterbury Accelerated Pavement Testing Indoor Facility (CAPTIF)

CAPTIF is located in Christchurch, New Zealand. It consists of a 58 m long (on the centreline) circular track, contained within a 1.5 m deep x 4 m wide concrete tank so that the moisture content of the pavement materials can be controlled and the boundary conditions are known. A centre platform carries the machinery and electronics needed to drive the system. Mounted on this platform is a sliding frame that can move horizontally by 1 m. This radial movement enables the wheel paths to be varied laterally and can be used to have the two "vehicles" operating in independent wheel paths. Elevation and plan views are shown in Figure 1.1.

At the ends of this frame, two radial arms connect to the Simulated Loading and Vehicle Emulator (SLAVE) units shown in Figure 1.2. These arms are hinged in the vertical plane so that the SLAVEs can be removed from the track during pavement construction, profile measurement, etc., and in the horizontal plane to allow vehicle bounce.

Figure 1.1 (a) Elevation view of CAPTIF.



(b) Plan view of CAPTIF.

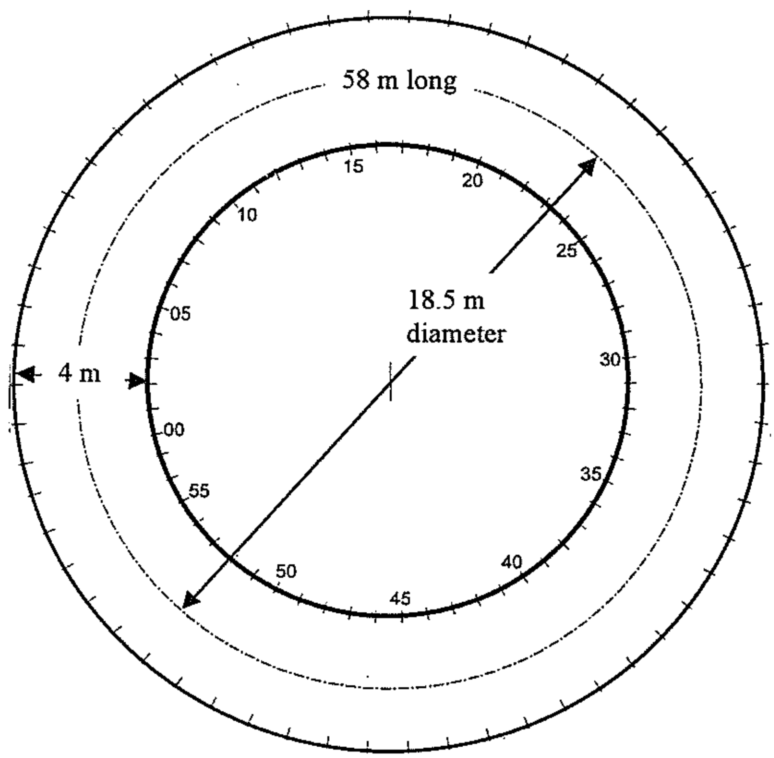
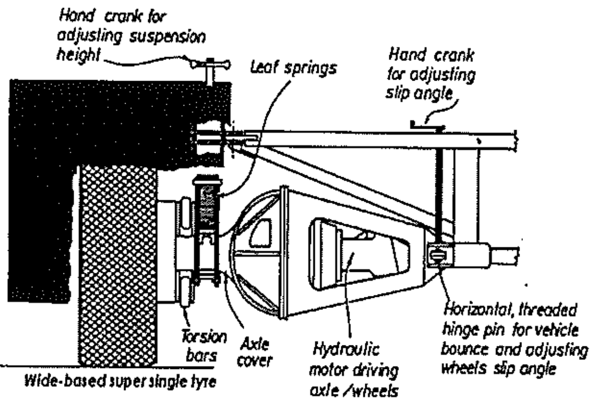


Figure 1.2 The CAPTIF SLAVE unit.



CAPTIF is unique among accelerated pavement test facilities in that it was specifically designed to generate realistic dynamic wheel forces. All other accelerated pavement testing facility designs that we are aware of attempt to minimise dynamic loading. The SLAVE units at CAPTIF are designed to have sprung and unsprung mass values of similar magnitudes to those on actual vehicles and use, as far as possible, standard heavy vehicle suspension components. The net result is that the SLAVEs apply dynamic wheel loads to the test pavement that are similar in character and magnitude to those applied by real vehicles. This was a significant factor in its selection for this project. Table 1.1 is a summary of the characteristics of the SLAVE units. The configuration of each vehicle, with respect to suspensions, wheel loads, tyre types and tyre numbers, can be identical or different, for simultaneous testing of different load characteristics.

Pavement instrumentation which is used at CAPTIF includes: Bison coil transducers for measuring vertical compressive strains in the lower layers of the pavement, H-bar strain gauges for measuring horizontal strains at the bottom of the asphalt layer, and partial depth gauges for measuring the pavement layer deflections. As well, temperature probes are used to monitor both the pavement and air temperatures. The vehicle instrumentation consists of accelerometers mounted on both the sprung and unsprung masses of each "vehicle" and displacement transducers to measure suspension displacements. As the "vehicles" are a fairly simple quarter vehicle structure, wheel forces can be calculated by combining the two accelerometer signals weighted by appropriate mass factors. Other measurement systems used at CAPTIF during testing are: a Falling Weight Deflectometer (FWD), the CAPTIF Deflectometer, which is a modified Benkelman beam, a DIPStick profiler, and a transverse profilometer. Recently a laser profilometer has been installed which supplements the DIPStick. For measurement convenience the track is divided into 58 equally spaced stations which are 1 m apart on the centreline wheel path. A more detailed description of the CAPTIF and its systems is given by Pidwerbesky (1995).

Table 1.1 Characteristics of a SLAVE unit.

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
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 wheel paths
Speed	0-50 km/h, programmable, accurate to 1 km/h
Radius of travel	9.2 m

1.3 Dynamic Wheel Forces and Pavement Wear

In the various studies on suspension dynamics the most widely used measure of dynamic wheel force is called the dynamic load coefficient (DLC) which is defined as:

$$DLC = \frac{\textit{standard deviation of wheel force}}{\textit{static wheel load}}$$

The DLC generated by a wheel traversing a pavement depends not only on the suspension and load configuration but also on the vehicle speed and the pavement profile. Sweatman (1983) proposed a standard test condition, which corresponds to highway speeds on a road of moderate roughness. For the suspensions he tested, he found DLC values ranging from 0.13 to 0.27 for this condition. While the results of other researchers cannot be compared directly these values are typical.

Previous efforts at converting dynamic wheel forces to estimates of induced pavement wear have been based on a number of assumptions. The most widely used approach is that developed by Eisenmann (1975), which assumes that the wheel force distribution is Gaussian and random, and that the fourth power law applies. On this basis a road stress factor, Φ, can be defined as:

$$\Phi = KP^4_{stat}\,[1 + 6\bar{s}^2 + 3\bar{s}^4]$$

where P_{stat} = mean axle load
$\bar{s}$ = coefficient of variation of dynamic wheel load ≡ DLC
K = constant

A dynamic road stress factor that reflects the contribution of vehicle dynamics to road stress can then be derived:

$$v = \frac{\Phi}{KP^4_{stat}}$$

$$\text{i.e. } v = 1 + 6\bar{s}^2 + 3\bar{s}^4$$

Applying this formula to the typical range of DLC values of the previous paragraph results in dynamic road stress factors approximately in the range 1.10 to 1.45. This suggests that, under those operating conditions, the worst suspensions would generate over 30% more road wear than the best.

The assumptions underlying the Eisenmann formula are, however, not correct. Although the wheel force distribution is approximately Gaussian and can be regarded as random in the time domain, wheel force measurement experiments have shown good repeatability for a particular vehicle at a given speed on the same test section of pavement. There is further evidence (Cebon 1987) that, even for different heavy vehicles, the peak loads tend to occur at the same approximate positions along the pavement.

More recently an experiment conducted as part of the OECD DIVINE programme (O'Connor et al. 1996, OECD 1998) recorded a pattern of spatial repeatability for dynamic wheel forces averaged over 3,000 heavy vehicles in the general fleet. Sweatman (1983) argued that it is the peak loads that are important in causing pavement failure and so proposed an alternative road stress factor using the 95th percentile dynamic loads. Applying the fourth power law to these this factor is given by:

$$\Phi_{95th} = (1 + 1.645\ DLC)^4$$

Using this equation with the DLC range presented previously, leads to dynamic road stress factors ranging from 2.17 to 4.35. Thus, based on this theory, the worst suspensions generate 100% more pavement wear than the best.

Both these factors assume a fourth power relationship between load and pavement wear, which, as mentioned in Section 1.1, is increasingly under question, particularly for New Zealand-style thin-surfaced pavements where it has never been properly validated. If a lower power value is used, the difference in the effect of different levels of dynamic loading decreases while, conversely with a higher power, the effect increases. These factors are also both derived quantities without any direct validation data.

The three pavement tests undertaken in this research programme were specifically aimed at determining experimentally the influence of dynamic wheel loads on pavement wear and pavement life. For two of the tests the particular focus was on the thin-surfaced pavement designs, which are predominant in the New Zealand roading network.

1.4 The Research Programme

Before the start of this research programme, the only suspension that had been used at CAPTIF was a traditional multi-leaf steel spring with no viscous damping. This type of suspension is typical of what was in use in 1986 when CAPTIF was designed and is still in common use.

For these tests and to enhance its capabilities, CAPTIF was modified to enable the SLAVE units to be fitted with more modern suspensions. Specifically, configurations using a steel parabolic leaf spring suspension, which has low inter-leaf friction and requires a viscous damper, and an air bag suspension also with a viscous damper, were designed and built.

The initial plan for this research programme was to compare a range of suspension configurations on a series of New Zealand-style thin-surfaced unbound granular pavement structures. Up to five tests were proposed with reviews after each. This plan was modified to include a test for Research Element 1 of the OECD DIVINE programme. Because DIVINE was an international co-operative research effort, the pavement design had to be acceptable to all the participating countries. For this reason an asphaltic concrete flexible pavement design was used. Although the use of this type of structure is limited to the very heavily trafficked parts of the road network in New Zealand, it is used extensively internationally and much of the research done on flexible pavements is based on this type of structure. One of the major benefits in testing this pavement design is that it provides a direct comparison with similar loads applied to a New Zealand pavement design in the subsequent test. As most international research work on flexible pavements is based on these asphaltic concrete structures, there is a great need to understand the limitations and issues when applying these findings to thin-surfaced structures.

Participating in this international project had a number of other benefits. The additional funding provided enabled a much more detailed and extensive instrumentation and measurement programme to be designed than would have been possible under purely local funding. A number of international experts were directly involved in the project providing additional expertise and advice. The international linkages and contacts established through this collaboration are continuing beyond the project and are of long-term benefit to New Zealand.

In Sections 2, 3 and 4 of this report, the test configuration, the experimental programme and an overview of the results are described for each of the three pavement tests that were conducted in turn. Section 5 then brings together the results of the three tests and presents the overall findings. Finally Section 6 draws conclusions from the results and makes recommendations for their implementation and use.

2. THE FIRST PAVEMENT TEST

2.1 Introduction

The modifications to the CAPTIF facility to accommodate different suspension configurations took longer than originally anticipated. Planning for the OECD DIVINE test was already underway when the first of the new suspension configurations, the parabolic steel leaf spring, became available for use. The initial plan had been that the first test would have the greatest possible difference in dynamic loading, which was expected to be from using the multi-leaf steel and air suspensions. However, in order to complete a first test and have the facility available for the DIVINE project, it was decided to begin with a comparison of the more modern parabolic steel suspension with the traditional multi-leaf steel spring. It was expected that the difference between the dynamic loads produced by these two suspension types would not be as great.

The pavement design used was a typical thin-surfaced flexible pavement. Based on the NRB (1989) design chart for a premium flexible pavement with thin surfacing, this had a design life of 350,000 ESA for a subgrade with a CBR value of 10-12% and a granular layer thickness of 250 mm. The SLAVE units were fitted with wide single tyres and loaded to 37 kN, which is approximately equal to half the reference load for an axle fitted with wide single tyres (7.2 tonnes). This means that one load cycle was approximately one ESA. Surprisingly, the pavement failed prematurely after only 35,000 load cycles. At this point the pavement profile was too rough to continue trafficking. In spite of this, some very significant results were obtained.

2.2 Test Configuration

The pavement design used for this test was intended to be a typical New Zealand thin-surfaced pavement structure. This consisted of 30 mm asphalt layer, over 250 mm of crushed rock basecourse, over a silty clay subgrade.

During construction the pavement was extensively monitored. Four arrays of Bison coil transducers, each of three coil pairs, were installed in each wheel path (from Stations 4-7 inclusive) to measure the vertical compressive pavement strains in the lower layers. Coil pairs were installed in the bottom 100 mm of the basecourse, the top 100 mm of the subgrade, and between a depth of 100 and 200 mm below the surface of the subgrade.On completion of construction the pavement structural condition was measured using the CAPTIF deflectometer. After construction, conditioning laps were applied to the pavement with both SLAVE units traversing the full width of the pavement in a uniform distribution. This is to simulate the normal post-construction compaction from traffic that occurs on in-service pavements.

For this test the SLAVE units were fitted with wide-based single tyres to maximise the wheel path separation. Lateral wander was set to ±100 mm with the nominal vehicle centrelines set at ± 350 mm from the track centreline. This is probably somewhat less than occurs in practice on in-service pavements but it was considered desirable to keep the two wheel paths as separate as practicable. The vehicle with the multi-leaf steel spring was operated on the outer wheel path while the vehicle with the parabolic spring and viscous damper ran on the inner wheel path.

After the conditioning laps, some basic measurements were undertaken to characterise the dynamic system. The first of these was the "EC bump test". The EC regulations for heavy vehicle drive axles include an additional load allowance for "road-friendly" suspensions. To qualify for this allowance the vehicle must be fitted with air suspension or "equivalent-to-air" suspension. "Equivalent-to-air" is defined as having a natural frequency of less than 2 Hz and damping greater than 20% of critical, with more than 50% of this damping being provided by a viscous damper. Three alternative testing methods are prescribed for measuring these quantities. One of these is to take the vehicle at crawl speed over a defined ramp that ends in an 80 mm drop and to measure the suspension response. One of these standard EC ramps has been constructed at CAPTIF and is used as a standardised repeatable test to characterise the suspension response. Figure 2.1 shows the EC bump test wheel force response for the two suspensions and loads used in this test. It is readily seen that neither of these suspensions meets the EC "equivalent-to-air" standard. Perhaps more surprisingly the parabolic leaf spring did not appear to be more road-friendly than the multi-leaf steel. It actually has a higher natural frequency.

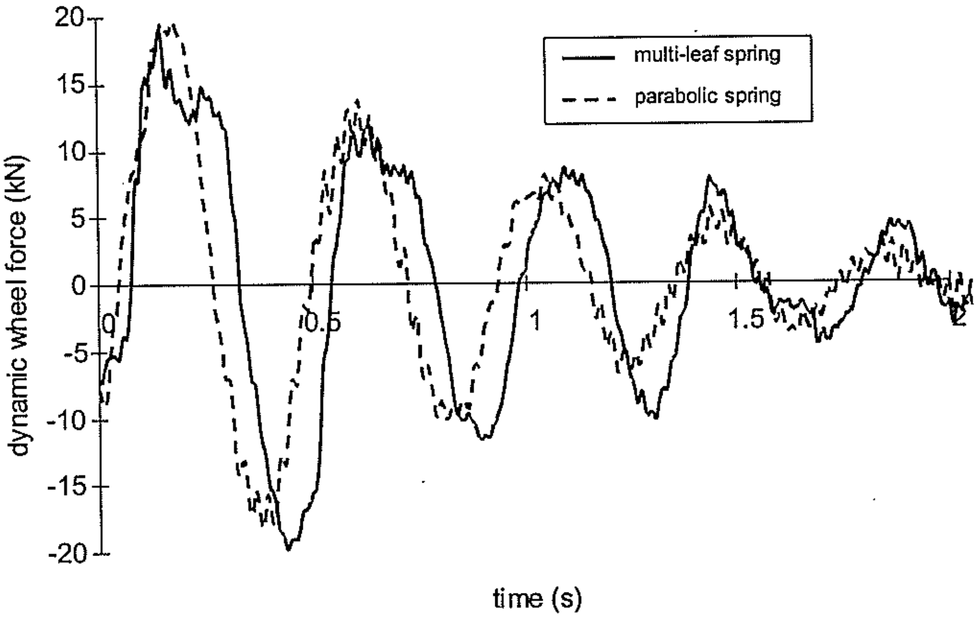


Figure 2.1 Comparison of suspension response to EC bump test.

However, the two suspensions did generate responses that were significantly different from each other. This is illustrated very clearly in Table 2.1, which shows the DLC values generated by the two vehicles on the same wheel path at different speeds. Although it had been intended that the parabolic suspension would be an example of a more road-friendly suspension, at the time no easy solution to improving its performance was seen. It was felt that the difference between the two suspensions was sufficient to show dynamic loading effects in the pavement performance, and the test proceeded on this basis. The reasons for the poor performance of the parabolic suspension are discussed further in Section 2.4.

Table 2.1 Dynamic load coefficient v speed for multi-leaf spring and parabolic spring.

Speed	Vehicle with multi-leaf spring	Vehicle with parabolic spring
20	0.126	0.084
45	0.120	0.164

2.3 Test Report

Pavement construction commenced in April 1993 with the excavation of the previous pavement. The subgrade layer was restored by back-filling. Testing of this layer was then undertaken. During April the AP20 basecourse material was manufactured. In May 1993 the basecourse layer was placed and tested. Construction was completed with the asphalt sealing.

Between May and August 1993, the design and fabrication of the new axle and the suspension assembly for the parabolic leaf spring was completed. It was originally intended that the first test would compare the multi-leaf steel spring with the air spring suspension as these were expected to show the largest differences. However, because of delays in the design and fabrication of the air suspension, it was decided to proceed with the first test using the parabolic steel suspension and the multi-leaf steel.

From August to December 1993, a series of measurements were undertaken to characterise the suspension performance and to debug the new measurement and data acquisition systems. During these measurements the trafficking of the pavement was distributed equally across both wheel paths to condition the pavement. This simulates the post-construction compaction that occurs in service. The measurements did not show the parabolic steel suspension as being more road-friendly than the multi-leaf steel spring. This was contrary to expectations and considerable effort was put into trying to resolve this.

By December 1993 plans were already well underway to conduct the second pavement test as part of the OECD DIVINE project. So it was decided to proceed with this first test so that it could be completed in time to begin construction of the DIVINE pavement. Although the two suspensions were not clearly differentiated in terms of road-friendliness they did generate significantly different dynamic load distributions. During the preliminary measurements 10,000 load cycles had been applied as conditioning laps. In early December 1993 the longitudinal and transverse pavement profiles and the pavement structural capacity were measured. Trafficking of the pavement commenced on 6 December 1993. After a further 10,000 load cycles, the profiles and wheel forces at 20 km/h and 45 km/h were measured. Trafficking resumed and was terminated on 12 December at 35,000 load cycles because the pavement roughness had deteriorated to the point where the suspension motions were excessive. This caused concern for the structural integrity of the facility and consequently for its safety.

The pavement profiles were measured again, as were the wheel forces, although the maximum speed was limited to 40 km/h because of concerns about safety and damage to the facility. Post-mortem excavations of the pavement were undertaken on 20 December 1993.

2.4 Results

In spite of the premature failure of the pavement and consequently the relatively small number of measurements, some quite significant results were obtained.

The longitudinal pavement profiles were measured along the centreline of each wheel path, mid-way between the two wheel paths and at 0.35 m inside the inner wheel path and 0.35 m outside the outer wheel path. Figures 2.2 and 2.3 show the profiles for the inner and outer wheel paths, together with the profiles of the adjacent wheel paths (the centreline profile is common to both plots). The elevation values are referenced to the concrete tank at zero and are in absolute units from sea level. Note that the profiles are plotted against angular position, not distance. This is done because, if the profiles are plotted against distance, profile features such as the peak at 310 degrees are not aligned for the different profiles. Thus the wavelength of these features is less for the inner profiles than for the outer profiles and these inner profiles will appear to be rougher. However this roughness is offset by the "vehicles" travelling more slowly in the inner wheel path than the outer wheel path.

For each of these profiles at each measurement interval, the International Roughness Index (IRI) was calculated. Figure 2.4 shows the progression of IRI for the two trafficked wheel paths. A number of points arise from this figure. The initial roughness of the two profiles is rather high. Rehabilitation might be considered for an in-service pavement having this roughness. This is offset by the vehicle speeds, which are somewhat lower than in-service highways. During the initial conditioning laps, the roughness decreases. This is the result of a smoothing of the short wavelength irregularities by the initial trafficking. Following this initial smoothing, the roughness stabilises with a very small increase over the next 10,000 load cycles. In the final 15,000 load cycles the roughness increases very rapidly, particularly on the inner wheel path, which was trafficked by the vehicle that was fitted with the parabolic leaf spring suspension. This change is associated with the formation of some severe rutting.

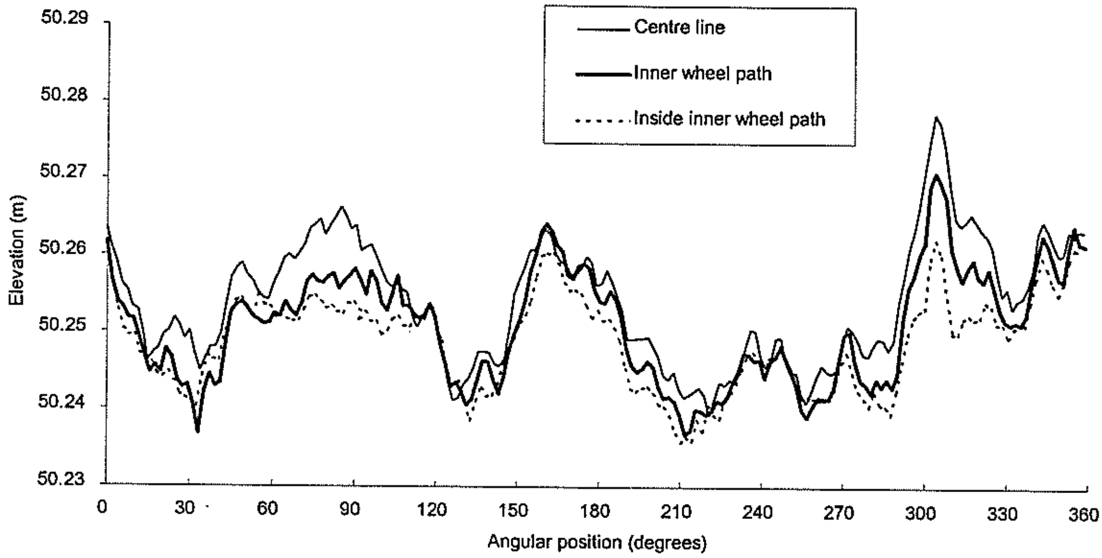


Figure 2.2 Profiles for inner wheel path and adjacent paths at ±0.35 m, immediately after construction.

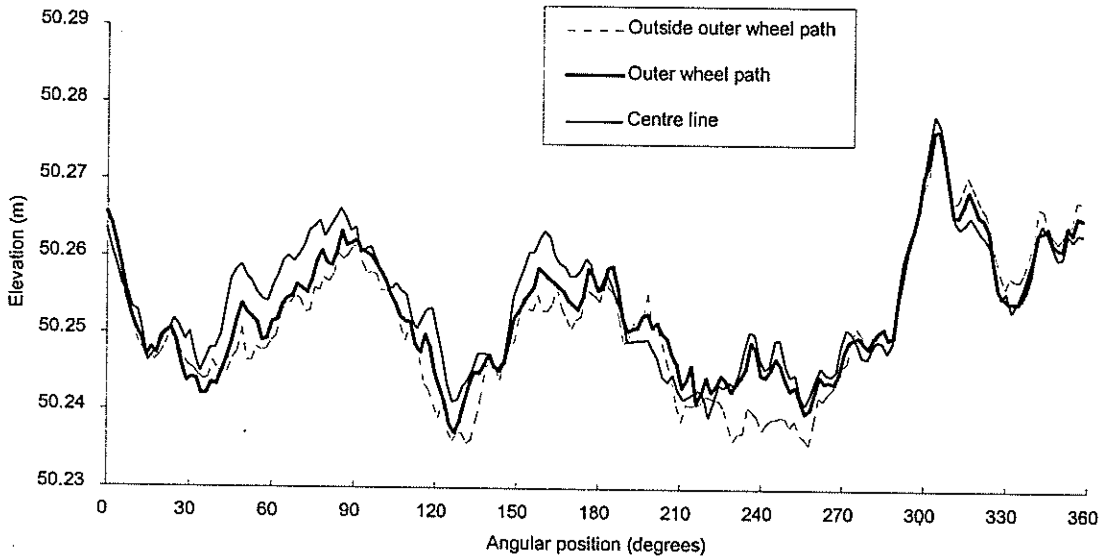


Figure 2.3 Profiles for outer wheel path and adjacent paths at ±0.35 m, immediately after construction.

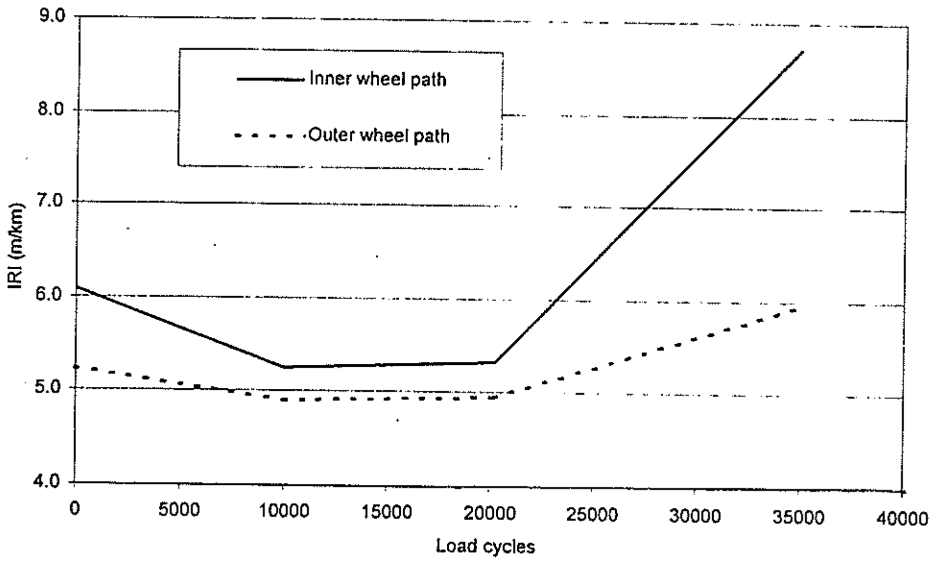


Figure 2.4 Change of IRI with trafficking for inner and outer wheel paths.

Figures 2.5 and 2.6 show the profiles at each of the loading measurement intervals, and they illustrate these points more clearly. During the initial conditioning between zero and 10,000 loads, a general compaction of the pavement of about 10-15 mm occurs, together with a smoothing of the short wavelength features. Between 10,000 and 20,000 load cycles there is much less change, particularly on the outer wheel path. Finally, in the interval between 20,000 and 35,000 loads, the formation of severe ruts at specific locations occurs, and the inner wheel path is the worst.

Figure 2.5 Longitudinal profiles for inner wheel path.

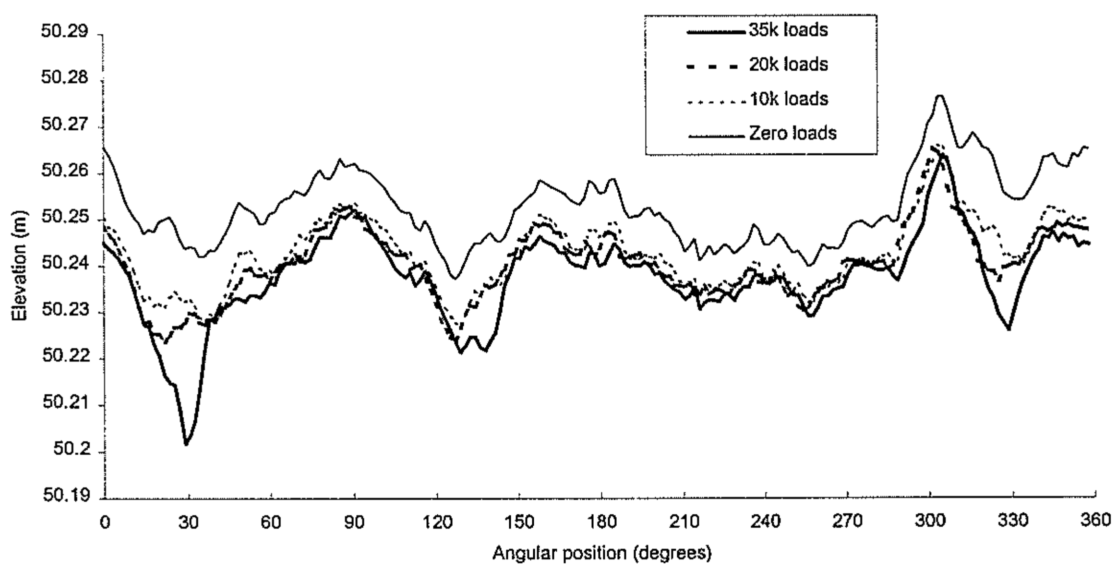


Figure 2.6 Longitudinal profiles for outer wheel path.

The transverse profiles were measured at each measurement interval. Figures 2.7 and 2.8 show the changes in transverse profiles at stations 2 and 4 around the track. As there are 58 stations around the track, these correspond to angular positions of 12.4 and 24.8 degrees respectively. From Figures 2.5 and 2.6, these positions are seen to correspond approximately to the locations of a major rut in each of the inner and outer wheel paths. Figures 2.7 and 2.8 show that these ruts appeared quite dramatically between 20,000 and 35,000 load cycles. Pavement material appears to have been shoved laterally leading to heaving adjacent to the wheel path.

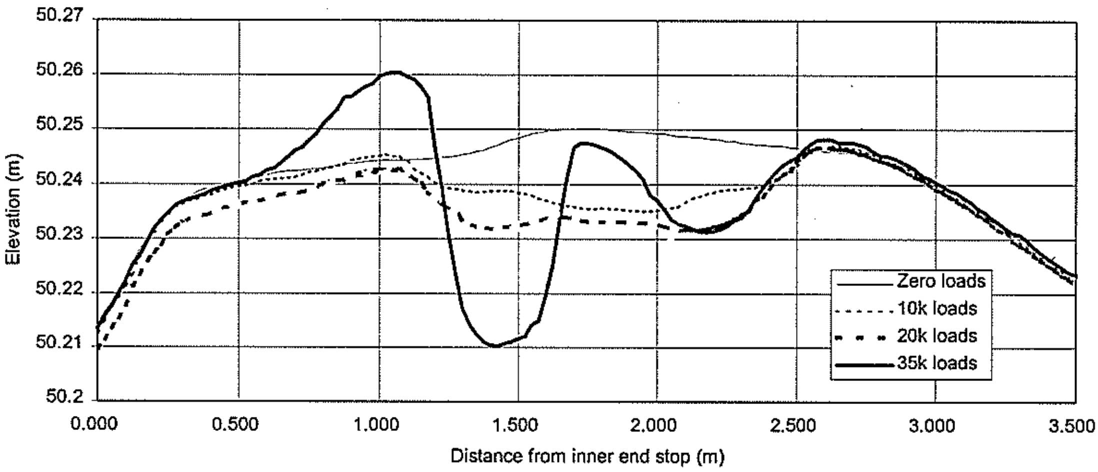


Figure 2.7 Transverse profiles at station 2.

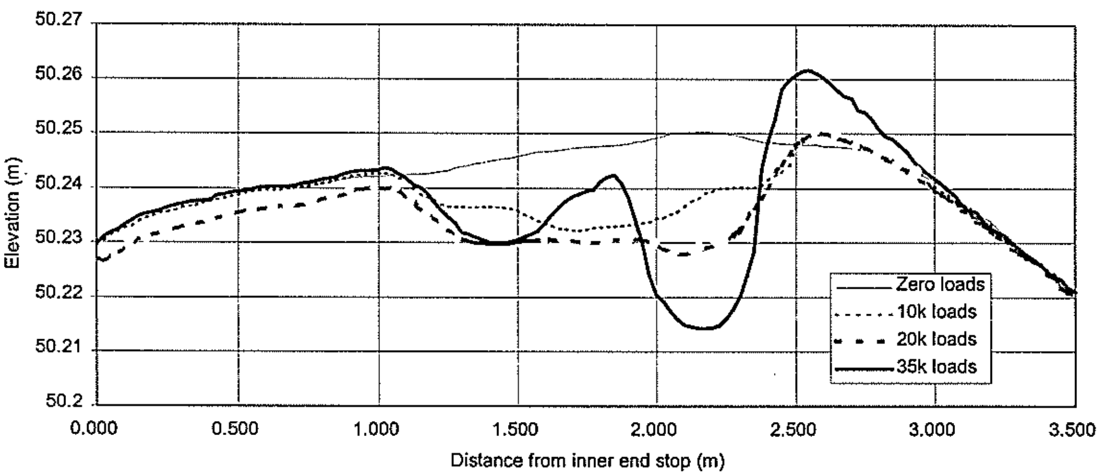


Figure 2.8 Transverse profiles at station 4.

Because the longitudinal and transverse profiles both measure the same quantity, i.e. the surface elevation, they can be cross-checked with each other. Figures 2.9 and 2.10 show a comparison of the longitudinal profiles, as measured using the Dipstick device, with the same longitudinal profiles re-constructed from the transverse profile measurements. As can be seen, the match is very good with the differences typically being 2 mm or less. The two methods of measurement do not take readings at exactly the same points on the pavement so a perfect match is not expected.

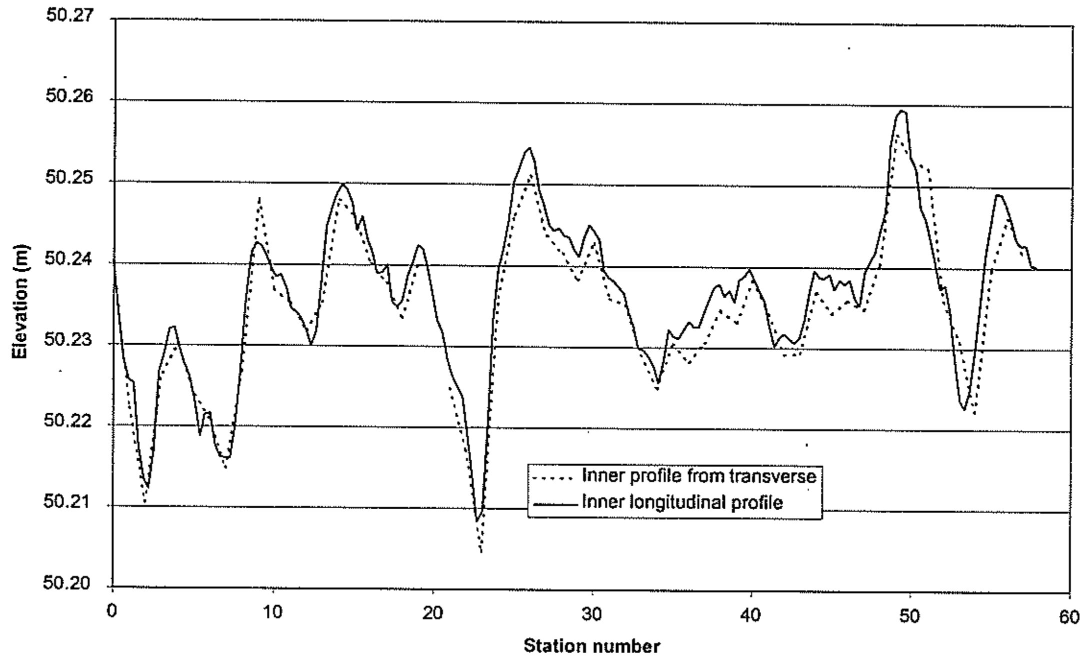


Figure 2.9 Comparison of longitudinal with transverse profiles on inner wheel path.

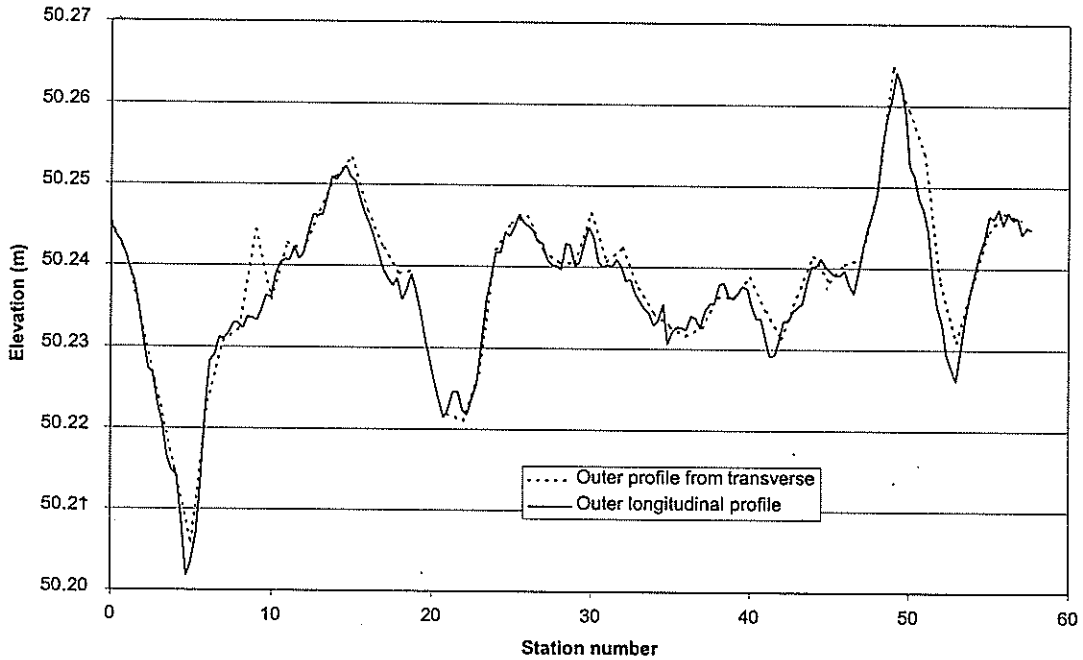


Figure 2.10 Comparison of longitudinal with transverse profiles on outer wheel path.

After the conditioning laps (10,000 load cycles), the CAPTIF deflectometer was used to measure the deflections of the pavement. By taking the difference of the profile measured at 35,000 loads from the profile measured at 10,000 loads, the change in profile can be determined. This change can then be compared with the structural capacity, as indicated by the peak deflections recorded by the CAPTIF deflectometer.

Figures 2.11 and 2.12 show these comparisons for the inner and outer wheel paths respectively. These figures show that the structural capacity of the pavement in the two wheel paths is very similar. In both cases the pavement is weakest (highest deflections) in a zone between station 55 and station 10. Note that the track is circular and that station 0 and station 58 are one and the same. As indicated by the markers on these figures the structural capacity measurements were not done at every station.

For the inner wheel path profile changes (Figure 2.11), substantial rutting peaks occurred at stations 2, 7, 23 and 53, while for the outer wheel path profile (Figure 2.12), the largest rutting peaks are at stations 5, 23, and 52. These locations are rounded to the nearest station. Although some of these can be broadly associated with a region of the pavement that is weaker, there is no indication in the deflection measurements of a weakness at station 23, for example (Figures 2.11 and 2.12). There is also no reason to expect the inner wheel path to be weaker than the outer.

From measurements of the vertical accelerations of the sprung and unsprung masses of the SLAVE units, the dynamic wheel forces applied to the pavement were calculated. These changed quite dramatically as the pavement roughness increased. Figure 2.13 shows the dynamic wheel forces generated by the SLAVE fitted with the parabolic leaf spring suspension on the inner wheel path at 20,000 load cycles and at 35,000 load cycles. Slightly different speeds were used for these two measurements because the pavement roughness after 35,000 loads was too severe for the SLAVE unit to operate safely at 45 km/h. The dynamic response of this parabolic leaf spring suspension showed that it was very poorly damped.

The comparable plot for the vehicle with the multi-leaf steel suspension operating on the outer wheel path is shown in Figure 2.14. This does not show the same increase in dynamic wheel forces between 20,000 and 35,000 load cycles but this is mainly because the outer wheel roughness did not increase to the same extent (see Figure 2.4). Interestingly, however, the spatial location of some of the wheel force peaks and troughs has changed.

Plotting these wheel forces on the same graphs as the longitudinal profile (Figures 2.15 and 2.16) shows that to a large extent the two are correlated. High dynamic wheel loads correspond to dips in the pavement profile and vice-versa. Although this relationship had previously been postulated it has never before been measured. The forces lag the profile very slightly, which is also as expected. Although this may seem intuitively obvious, it is not as straightforward as it seems. The wavelength of the vehicle response is primarily a function of the characteristics of the vehicle dynamics (mass and suspension stiffness) and of its speed, rather than of the excitation provided by the pavement profile.

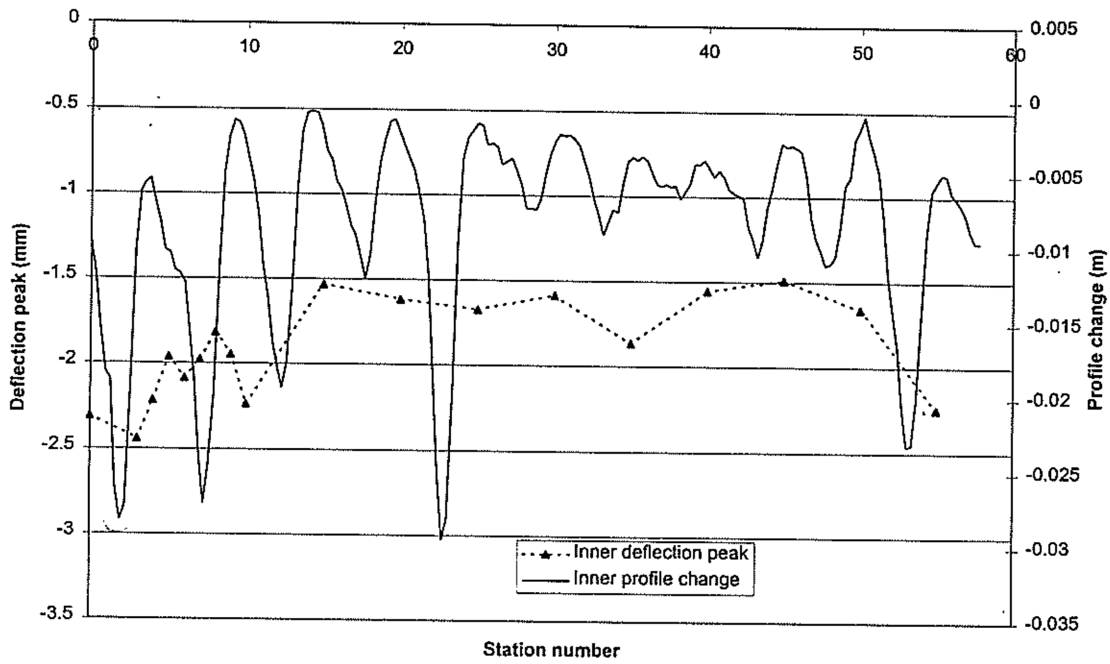


Figure 2.11 Comparison of profile change against structural capacity for inner wheel path.

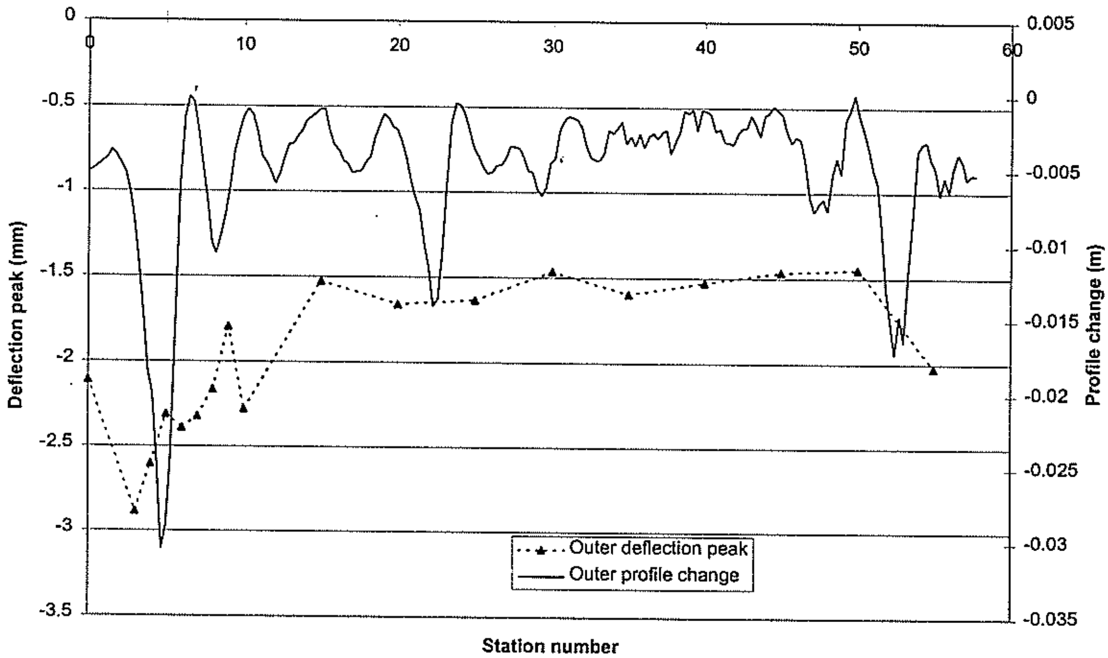


Figure 2.12 Comparison of profile change against structural capacity for outer wheel path.

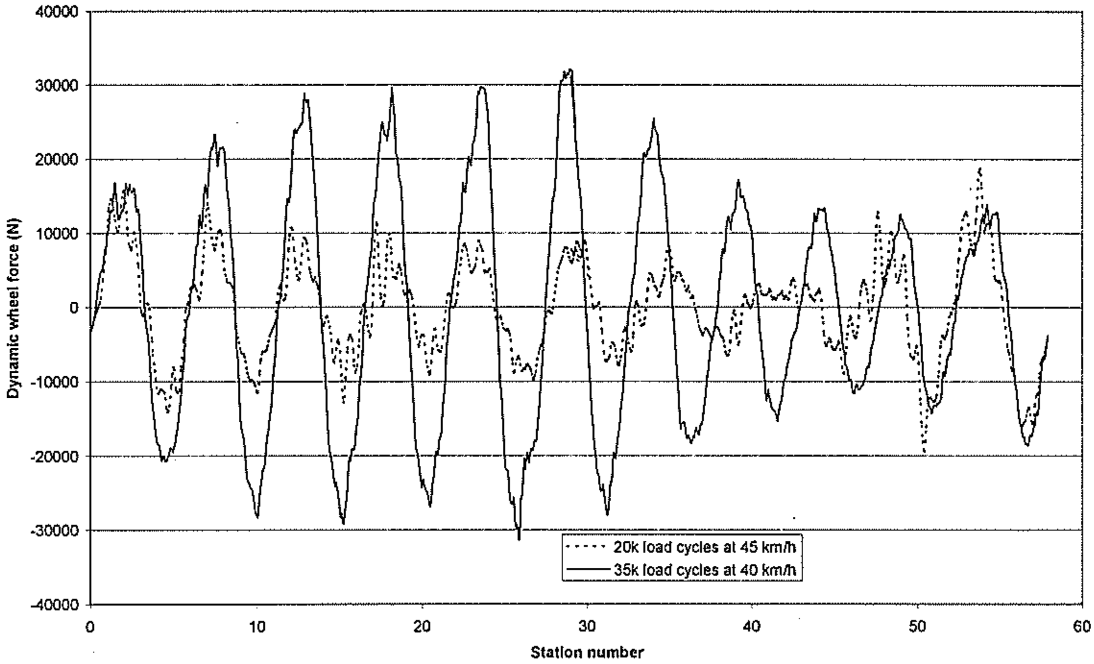


Figure 2.13 Dynamic wheel forces from parabolic leaf spring suspension on inner wheel path.

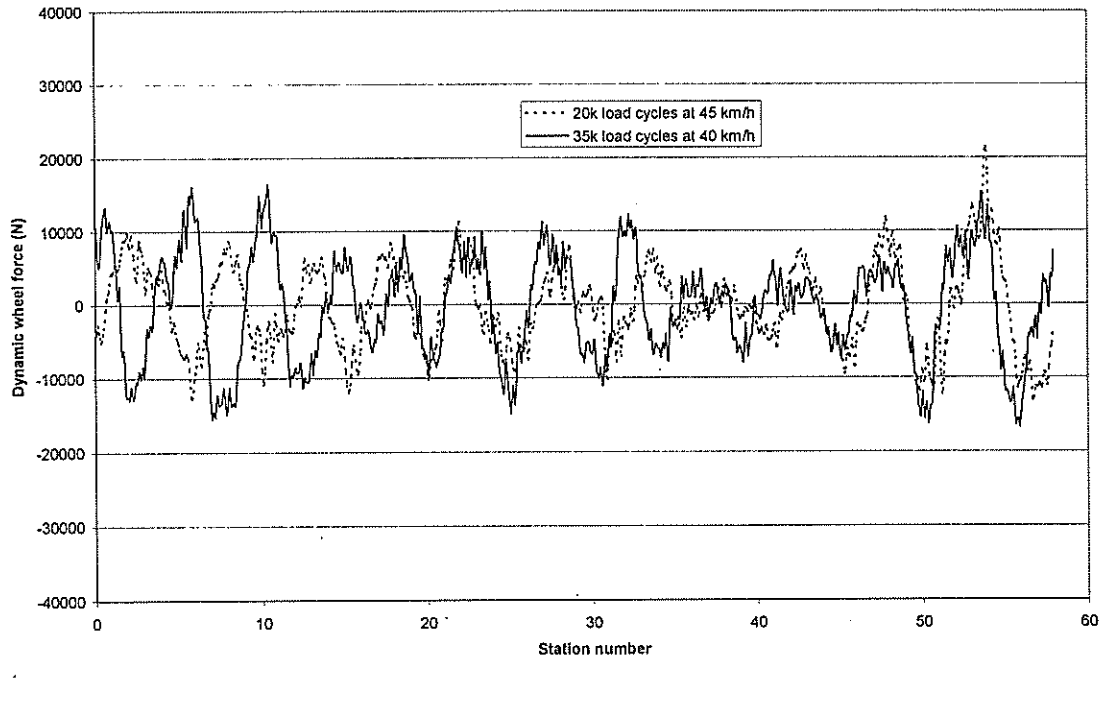


Figure 2.14 Dynamic wheel forces from multi-leaf spring suspension on outer wheel path.

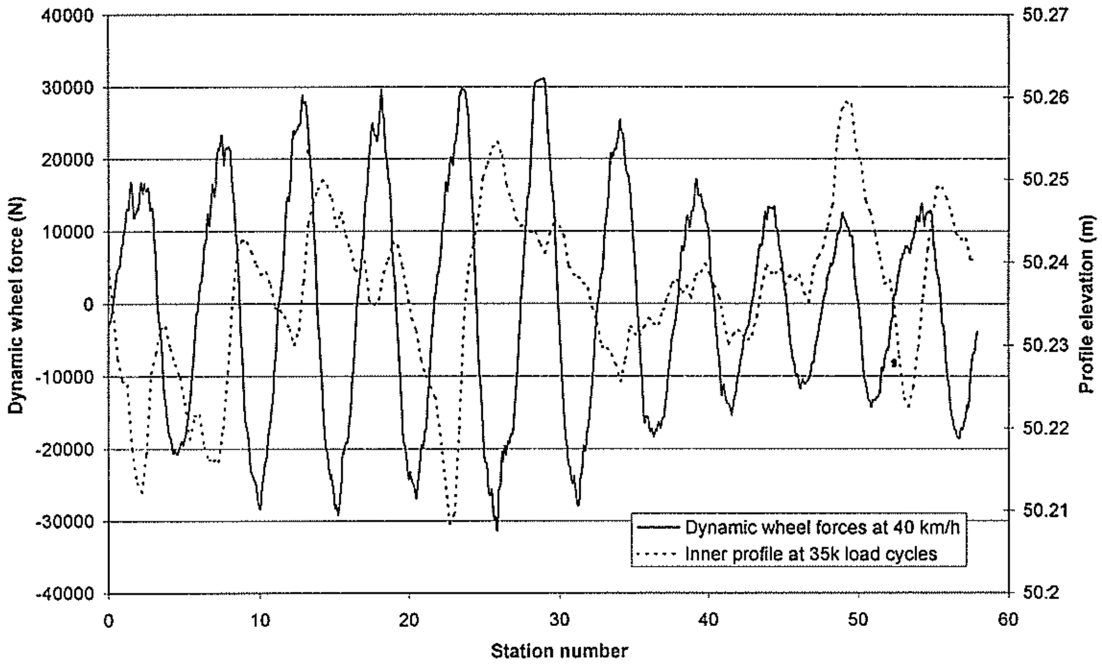


Figure 2.15 Comparison of dynamic wheel forces with pavement profile on inner wheel path.

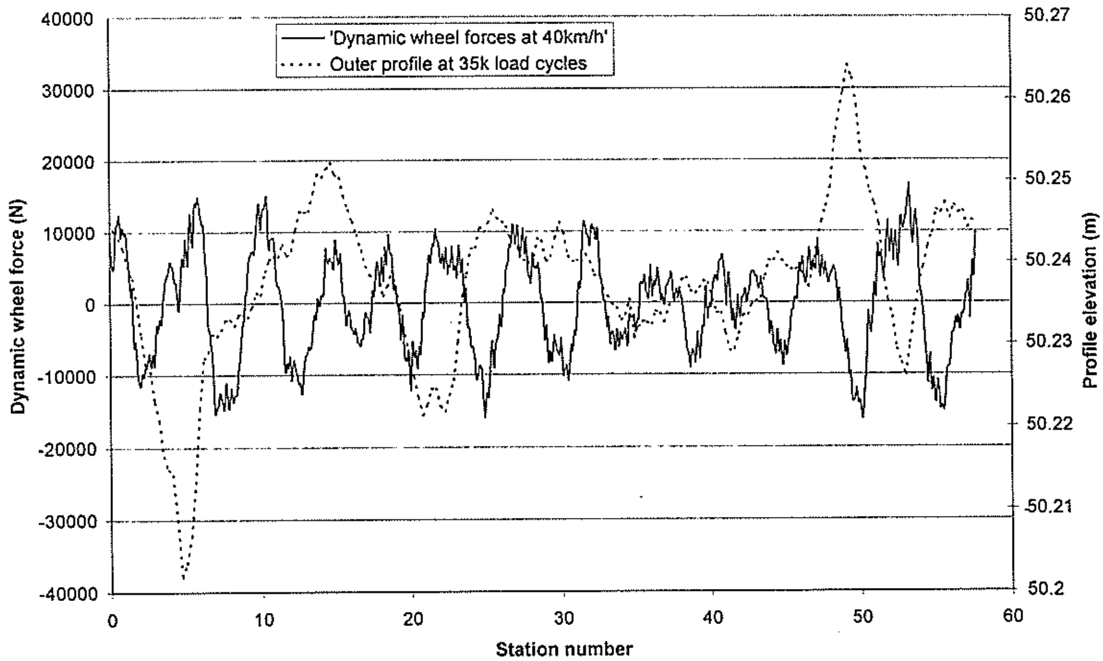


Figure 2.16 Comparison of dynamic wheel forces with pavement profile on outer wheel path.

This suggests that the pavement profile has to a degree been determined by the dynamic loads being applied. Clearly this is not the only factor because the correlation between forces and profiles is not perfect. The initial profile determines the initial dynamic wheel force pattern, and construction variability leads to local "weaknesses" in the pavement structure that influence profile changes.

These relationships can be quantified by calculating the normalised cross-correlation functions between the wheel forces and the profiles. Figures 2.17 and 2.18 show plots of these functions for the inner and outer wheel paths. Peak correlation values are ± 0.47 for the inner wheel path trafficked by the vehicle with the parabolic leaf spring suspension, and ± 0.33 for the outer wheel path trafficked by the vehicle with the multi-leaf steel spring suspension. These correlations are far from perfect as perfect correlation would result in values of ± 1. However, the periodic nature of the functions does indicate a relationship. As expected the first correlation peak is negative (a dip in pavement profile corresponds to a peak in wheel force and vice-versa) and has a small lag (the force variations occur a little downstream of the profiles). The second correlation peak in both figures is positive and just as large if not slightly larger than the first peak. This shows that half a wavelength downstream of a profile feature there is a matching wheel force response in the same direction.

For the inner wheel path and the parabolic suspension (Figure 2.17), the magnitude of this correlation does not start to decay until after the third peak which is indicative of the very low level of damping in this suspension. For the outer wheel path and the steel suspension (Figure 2.18), the correlation decays much more rapidly showing the higher degree of damping in this suspension.

To investigate this further the dynamic wheel forces can be compared with the profile change rather than just the absolute profile. These are shown in Figures 2.19 and 2.20. The correlation of high dynamic wheel forces with negative changes in pavement profile (rut formation) is even greater here, particularly for the inner wheel path which was trafficked by the vehicle with the parabolic leaf spring suspension. This suspension is more linear in its response than the multi-leaf steel. As the magnitude of the dynamic wheel forces increased, the resonant frequency of its response does not change. This is illustrated in Figure 2.13.

The response of the multi-leaf steel spring is more complex because of the inter-leaf friction. For small displacements the spring is very stiff. Once the static friction is overcome the stiffness reduces, and for larger displacements the spring is much less stiff. (For a more detailed analysis of the behaviour of steel leaf springs the reader is referred to Fancher et al. (1980).) The effect of this is that the vehicle fitted with the multi-leaf spring appears to have a range of resonance modes which are amplitude dependent. Nevertheless there is still an obvious correlation between dynamic load peaks and profile changes for this vehicle.

As with the previous case we can determine numerical values for the correlation by calculating the normalised cross-correlation functions. These are shown in Figures 2.21 and 2.22.

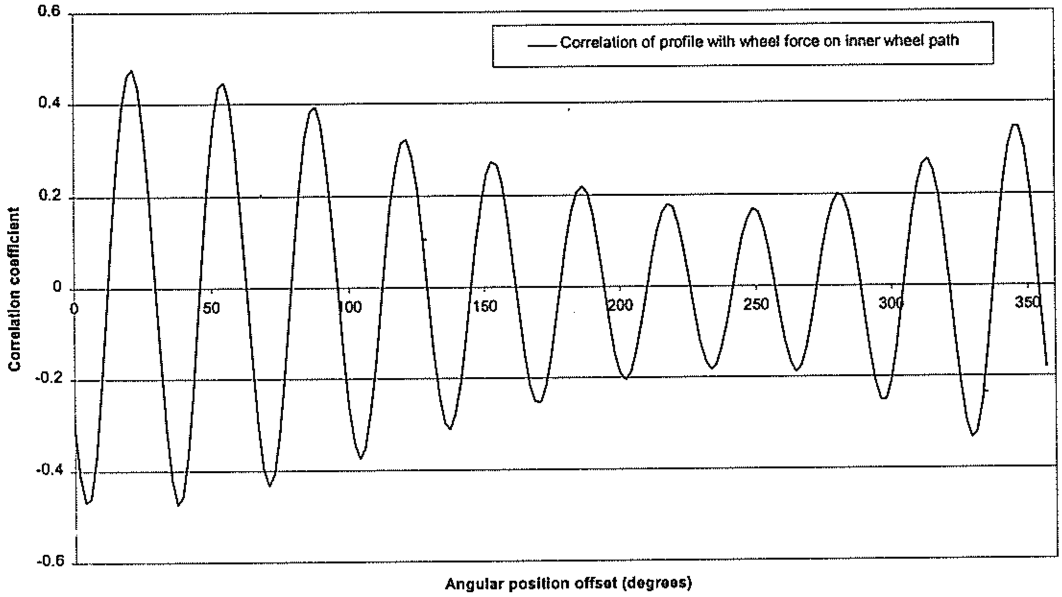


Figure 2.17 Cross-correlation of profile with wheel force for inner wheel path and parabolic leaf spring suspension.

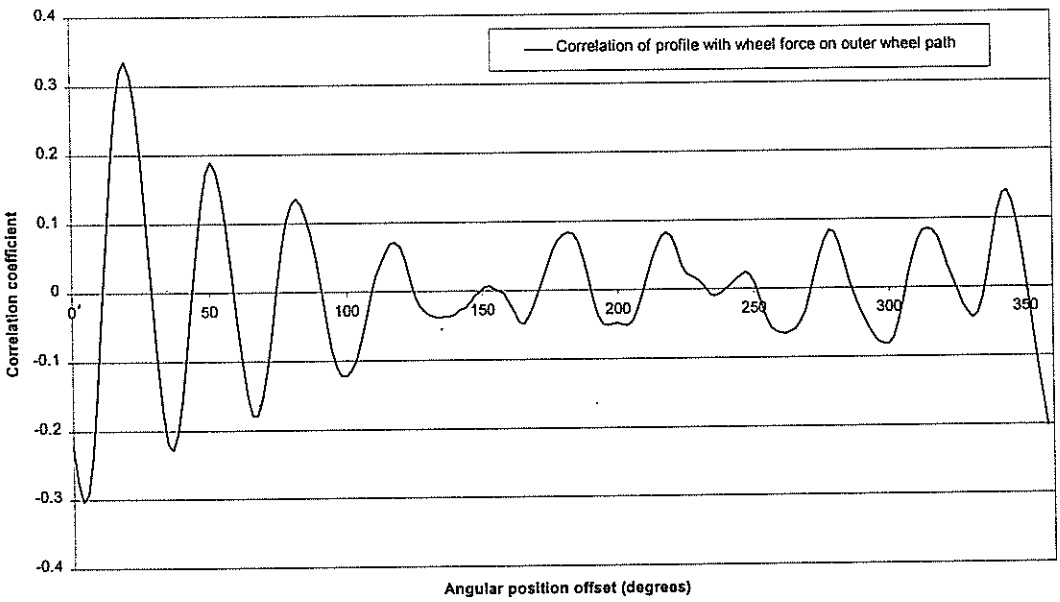


Figure 2.18 Cross-correlation of profile with wheel force for outer wheel path and multi-leaf steel spring suspension.

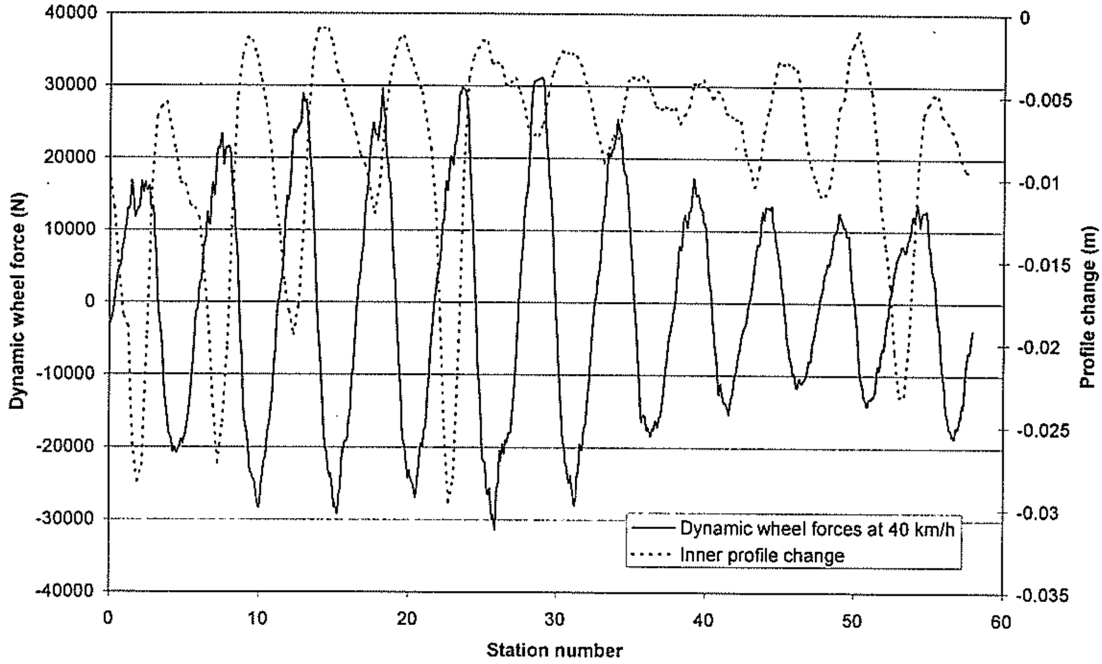


Figure 2.19 Comparison of dynamic wheel forces with profile change on inner wheel path.

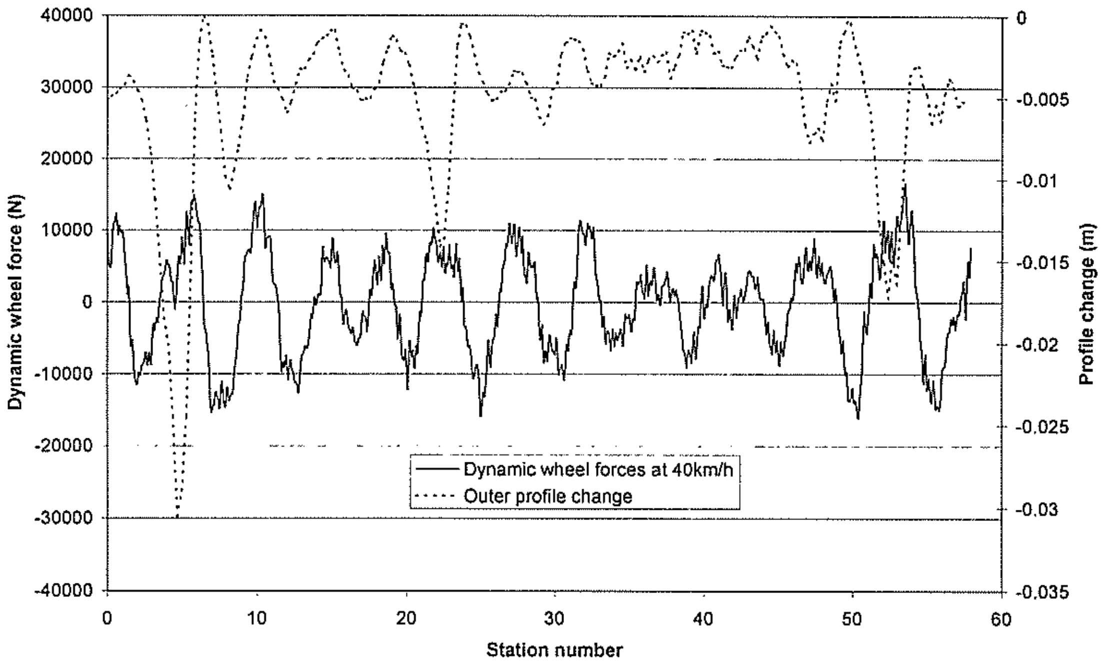


Figure 2.20 Comparison of dynamic wheel forces with profile change on outer wheel path.

In form these correlation functions are very similar to those shown in Figures 2.17 and 2.18 except that the magnitudes are somewhat higher, with peak values for the inner wheel path of 0.69, and for the outer of 0.48. This indicates that the correlation between the wheel force and the profile change is stronger than the correlation between the wheel force and the profile itself. Although the dynamic wheel forces are generated from excitations by the vehicle passing over the profile, the dynamic characteristics of the vehicle itself are an important factor and do not depend on the profile. On the other hand the change in profile is determined by loads applied and the pavement strength. If the pavement strength is uniform then the dynamic vehicle loads will be the dominant factor. As with Figures 2.19 and 2.20 the poor damping of the parabolic leaf spring and the higher level of correlation with this suspension are clearly visible.

2.5 Conclusions

In this test the pavement failed prematurely achieving only about 10% of its design life. The mode of failure was through rutting of the basecourse layer, with a considerable volume of material being shoved laterally. The cause of this problem appears to have been inadequate compaction of the basecourse layer, which was attributed to the performance of the vibratory roller on a circular track where the roller is constantly being turned. Subsequently a new flat plate compactor known as a Wacker has been acquired and its use at CAPTIF has given improved results.

The test compared the effect of the dynamic loads produced by two different suspension types on pavement performance and life. Although the parabolic leaf spring with viscous damping is a more modern design and was expected to be more road-friendly, this was not the case. In fact the wheel path trafficked by the vehicle fitted with this suspension suffered greater damage.

There are two fundamental reasons why the parabolic leaf spring suspension was not more road-friendly than the multi-leaf steel spring. The first is that the spring was too stiff resulting in a high natural frequency, and the second is that the suspension was not adequately damped. These deficiencies were recognised before the start of the test from the results for the EC bump test results (Figure 2.1) but could not be investigated in depth until after the test was completed.

The spring was too stiff because it was selected to match the design load of the new axle fitted to the SLAVE unit. This was designed to generate a 6 tonne wheel load (i.e. 12 tonne axle) with a sprung mass component of approximately 5.2 tonnes. For this test the wheel load was 3.7 tonnes with a sprung mass of about 2.9 tonnes and so the spring was clearly too stiff by a factor of nearly 2. This overly stiff spring was also a factor in the damping as it increases the value of the critical damping level. The actual damping provided by the shock absorber was therefore a smaller proportion of the critical value than it would be with a softer spring.

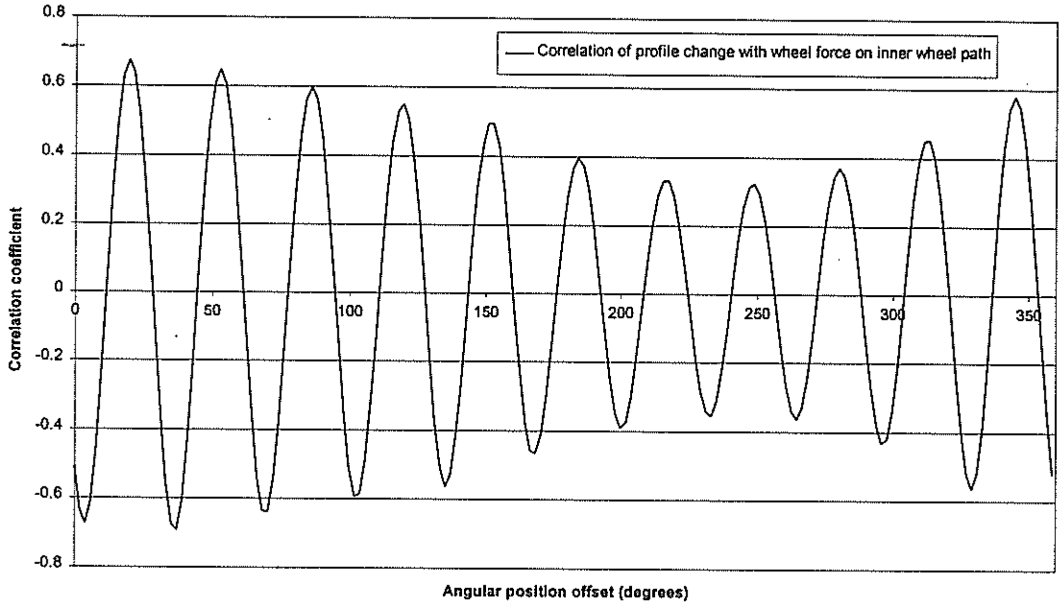


Figure 2.21 Cross-correlation of profile change with wheel force for inner wheel path and parabolic leaf spring suspension.

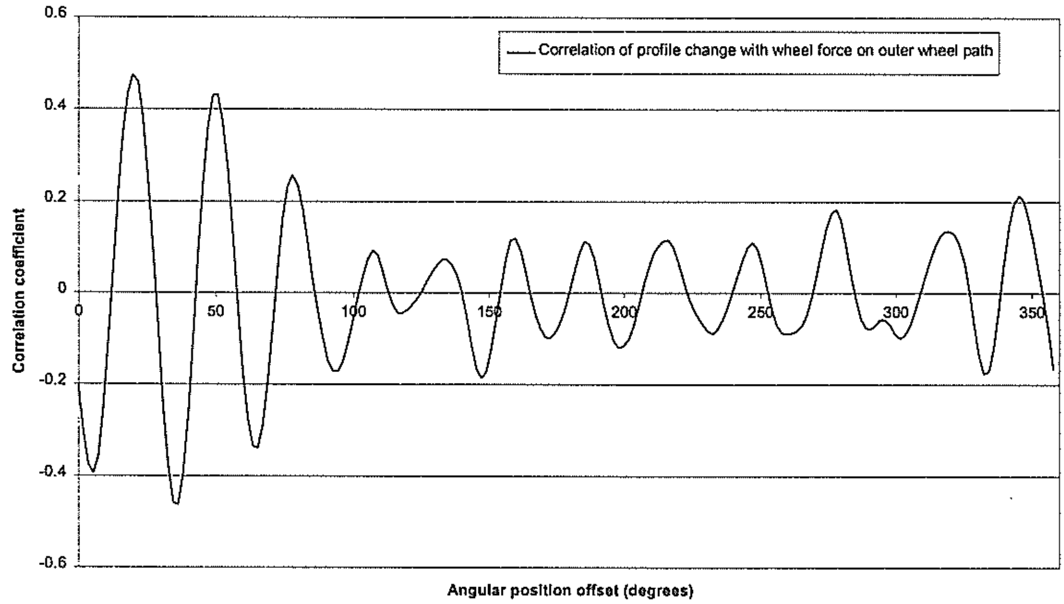


Figure 2.22 Cross-correlation of profile change with wheel force for outer wheel path and multi-leaf steel spring suspension.

The parabolic spring used at CAPTIF consisted of two leaves and a modification has been developed where the spring is disassembled and only one leaf is used. This allows a more road-friendly steel suspension to be configured with lighter axle loads.

The problems with this overly stiff suspension raise some interesting issues related to in-service vehicles in New Zealand. New Zealand has relatively low axle load limits by international standards. However, our heavy vehicle prime movers are all sourced overseas, many from countries allowing higher axle load limits than here. Often these axles and suspensions are designed for significantly heavier loads than legally permitted in New Zealand. Although air suspensions regulate their stiffness in response to the load carried, most steel suspensions do not and so the suspension is stiffer than necessary. This results in higher dynamic forces and poorer ride quality. The problem may well be exacerbated by our road user charges regime which encourages the use of vehicle configurations with even lower axle loads. For the operator, having over-designed suspensions may seem attractive because they may be more robust and durable. Lowering axle loads does reduce pavement wear even with a suspension that is too stiff. However by better matching the suspension stiffness to the load, further improvements in both dynamic wheel forces and ride are achievable.

The high level of dynamic forces associated with inadequate damping is a fundamental issue. This has previously been identified by de Pont (1996) and Hahn (1985). Even a soft compliant suspension ceases to be road-friendly if it is inadequately damped. This can occur through poor suspension design or through damper deterioration.

Although the premature failure of the pavement limited the amount of data collected, some useful insights were obtained. From the structural capacity measurements a section of the pavement appeared to be weaker and some, though not all, of the rutting peaks occurred in this region. There was recognisable correlation between the dynamic wheel loads and the underlying pavement profile. While this has been postulated in the past, this was the first time this had ever been measured anywhere in the world. Because the vehicle response is as much a function of the vehicle characteristics as the underlying profile, this correlation indicates that the pavement profile was formed in response to the loads. This is further reinforced when the relationship between the change in profile and the dynamic wheel forces is considered. The correlation in this relationship is even stronger. Although it is difficult to quantify the effect, because of the relatively few measurement sets taken during the test, there is good evidence of a relationship between dynamic wheel forces and permanent deformation of the pavement.

3. THE SECOND PAVEMENT TEST - OECD DIVINE

3.1 Introduction

In 1990 the OECD initiated a Scientific Expert Group called IR2 to undertake an investigation into the dynamic loading of pavements. The group, comprising experts from 15 OECD member countries, worked together for two years to produce a state-of-the-art report which was published in November 1992 (OECD 1992). One the major findings of the OECD study was the identification of a number of critical areas needing further research. Because of the scale of this work and its broad applicability, the study recommended that this research be undertaken as an international co-operative effort. Accordingly the OECD established a new Scientific Expert Group, IR6, to undertake this co-ordinated research programme. This programme, which became known as DIVINE (Dynamic Interaction Vehicle and INfrastructure Experiment), consisted of six inter-related research elements. Research Element 1 was an accelerated pavement test designed to determine the extent to which differences in dynamic loading related to different suspension types influences pavement performance.

The key requirement for the accelerated pavement testing facility needed for this Research Element was that it be capable of applying dynamic loads to the pavement. A sub-committee of the IR6 group evaluated a number of facilities including those at Nantes in France and Madrid, Spain, before eventually recommending CAPTIF. Although there was some resistance to the choice of CAPTIF because of its geographical remoteness, it was the only accelerated pavement test facility specifically designed to generate realistic dynamic loads. Thus it was the only facility capable of undertaking the experiment without substantial modifications. The other major factor in favour of CAPTIF was the strong multi-disciplinary support team available in New Zealand.

Experimental design was undertaken by consensus of the IR6 Scientific Expert group, which included representation from most of the OECD member countries. The result of this approach was a fairly intensive set of measurement requirements and a pavement design that was a compromise between the thicker asphaltic concrete pavements used on the major European motorways and the thin-surfaced structures widely used in Australasia. The need to achieve the failure criteria within a reasonable time (and hence cost) was a factor in favour of a thinner pavement structure than might otherwise have been the case.

3.2 Test Configuration

3.2.1 Pavement Design

Initially CAPTIF staff produced a pavement design, in consultation with the IR6 analysis group, using the Transit New Zealand design method (NRB 1989) which designs for equal probability of rutting or cracking failure and then increases the asphalt thickness by 10%. Based on a silty clay subgrade layer with an in-situ CBR of 12, this design consisted of a 150 mm-thick basecourse layer of 20 mm crushed rock, and a surface layer of 80 mm of asphaltic concrete. The nominal design life of this pavement is 1.5 million ESA. In New Zealand the reference weight for an axle with wide single tyres is 7.2 tonnes. Therefore, with the CAPTIF vehicles on wide single tyres loaded to an axle weight of 10 tonnes, each load cycle would correspond to 3.72 ESA. Thus the expected design life of this pavement would be approximately 400,000 rig cycles which, at normal operating speeds, corresponds to approximately 550 hours of actual testing. Bearing in mind that pavements at CAPTIF are protected to some degree from environmental effects and that this usually results in extended pavement life, this was considered a realistic design life target.

The IR6 group felt that increasing the asphalt thickness to say 100 mm and the basecourse thickness to say 200 mm with a compensatory weakening of the subgrade would enhance the credibility of the pavement with the pavement engineering community when interpreting the results. However, the experience of CAPTIF staff is that there are practical difficulties in constructing a pavement with a thin basecourse layer on a weak subgrade (CBR < 10) in the CAPTIF environment, and thus the option to weaken the subgrade was limited. The final design specification was a 1220 mm silty clay subgrade layer with an in-situ CBR of 10-12, overlaid with a 200 mm crushed gravel (maximum particle size 20 mm) basecourse layer, and surfaced with an 80 mm layer of asphaltic concrete.

The pavement thicknesses adopted for the CAPTIF test pavement were a compromise between Australasian and European/North American designs. The aim was to provide a reasonable life of about 500,000 load applications of the CAPTIF vehicles. Using the selected pavement design, New Zealand empirical design practice at the time (1993) gave a fatigue life of approximately 1 million loading applications, and a deformation life of 2 million load applications. However, the New Zealand design procedure had never been verified for such a pavement and with these loading conditions. Major unknown factors were the effect of the concentrated wheel path distribution and the lack of adverse environmental influences. Several members of the IR6 group undertook modelling analyses to predict the pavement life and obtained considerably lower values. For example, de Boissoudy (1993) calculated a deformation life ranging from 169,000 load cycles to 1,450,000 load cycles depending on which strain-life model he used. The longer life value derives from the NAASRA model used in the AUSTROADS guide (AUSTROADS 1992). Post-construction analyses using measured material properties predicted even shorter life values.

3.2.2 Vehicle Configuration

For the test, one SLAVE vehicle was fitted with the original steel suspension unit. This is a two-stage trapezoidal multi-leaf steel spring. Damping for this suspension is generated by the interleaf friction of the spring. It has no viscous dampers fitted. The other vehicle was fitted with an air spring suspension with damping provided by a viscous damper. These two configurations represent opposite ends of the road-friendliness spectrum in terms of what is currently known about heavy vehicle suspension performance.

The following target performances were set. First, using the EC bump test described in Section 2.2, the stiffness and damping parameters were to be determined using the suspension deflection response. The "good" suspension had to meet the requirements of having a natural frequency less than 2 Hz and a damping rate of greater than 20% of the critical damping value, with at least 50% of the damping being provided by a viscous damper. The "poor" suspension had to fail these criteria.

Second, the wheel forces of both vehicles in the same wheel path on the newly constructed pavement were to be measured at a steady speed of 45 km/h and the dynamic load coefficient (DLC) values were to be calculated. The aim was that the "poor" suspension should have a DLC value double that of the "good" suspension.

Tyre configuration (single or dual), lateral wander and wheel path separation are three factors that interact with each other at CAPTIF. For example, increasing lateral wander reduces wheel path separation and thus, although desirable, it is not possible to maximise both these factors. IR6 considered that representative failure mechanisms would occur with only minimal lateral wander, and that a higher priority should be given to maximising separation in order to minimise the interference between the two wheel paths. Thus wide single tyres (385/65 R 22.5) operating at a pressure of 700 kPa were used for the tests with a lateral wander of +/-50 mm. Static wheel loads were 5 tonnes.

3.2.3 Construction Measurements

An extensive measurement programme was specified for this test. Before construction a series of tests was conducted to provide material property information for design and construction purposes. The subgrade and basecourse materials were subjected to the Proctor test to determine optimum moisture content and densities. These tests were undertaken at five different moisture content levels and repeated several times. From these results recommended values for mean and standard deviation tolerances on moisture content and in-place density were determined. For the asphaltic concrete, the Marshall method was applied to design the mix for optimum binder content, voids and density.

A second series of pavement materials tests relating to pavement performance and primary response was then undertaken. The primary purpose of this series was to characterise the pavement materials for analysis. These data were used as input to models and are important in applying the test results beyond the confines of the CAPTIF facility. In this category of tests, Canterbury University conducted repeated load tri-axial tests on both the subgrade material and the unbound basecourse material. Each material was tested at three different moisture contents in accordance with AASHTO test procedure T292 (1993) and then to 100,000 cycles to determine rutting. The outputs of these tests were the modulus and rutting parameters for the materials. The asphaltic concrete was subjected to dynamic creep, static creep, indirect tension and fatigue tests, which were conducted by ARRB Transport Research at their Melbourne laboratory.

To enable future cross-referencing of the test results with other laboratories, samples of the pavement materials were required to be kept for three years from completion of the test. These are in storage at CAPTIF.

The construction of the pavement was monitored intensively. The fines and crushed gravel were placed in lifts each not exceeding 150 mm in thickness. At the completion of each lift the density was measured at 1 m stations along three lines, which were the two wheel paths and a point midway between, using the probe in back-scatter mode (i.e. without penetration into the material). At the top of the subgrade and basecourse layers, 12 sets of readings were taken where the densities were measured at 0.5 m intervals transversely, across the full 4 m width of the track. An important requirement was that longitudinal variability be strictly controlled and that transverse variability be kept to an absolute minimum. The large number of transverse measurements was aimed at demonstrating this.

Each lift was compacted to 100% ± 4% of normal dry density. For the first lift of each material the density was measured at 5 m stations along the centreline after each complete pass of the roller, to confirm the density-compactive effort relationship. For the subsequent lifts of each material the density was measured similarly after every three complete passes of the roller. Densities were measured by nuclear density meter and CBR values were inferred from dynamic cone penetrometer tests. (Previous trials at CAPTIF have determined the relationship between dynamic cone penetrometer results and CBR values for the subgrade materials.)

The material was compacted using a vibrating steel roller, and the levels of each layer were determined using a profile beam with measurements being taken every 1 m interval circumferentially, and at 0.2 m intervals radially. The basecourse was placed so as to avoid segregation, and the basecourse surface was compacted using a pneumatic tyre roller. The asphalt surfacing was placed by hand in two equal lifts.

To monitor the transverse variability of the layers, a variable equal to the difference in thicknesses between wheel path centres was calculated at each station for each layer.This difference variable was required to have a mean less than ±3 mm with a standard deviation of less than 11 mm for the crushed rock basecourse layer (the maximum particle size of this material is 20 mm), and a mean of less than ±0.8 mm with a standard deviation of less than 3 mm for the asphalt layer (the maximum particle size in this layer is 7 mm). The tolerances of the mean values were based on the 95% confidence interval for a mean of zero, given the standard deviations quoted and 58 sample points. A much greater degree of longitudinal variability was acceptable though not necessary.

The IR6 analysis group also decided that falling weight deflectometer (FWD) tests should be used to monitor the pavement's structural capacity during construction. This was done at the top surface of the subgrade and then for the crushed rock and asphalt layers. These tests were undertaken by a local engineering consultancy (Tonkin & Taylor). FWD measurements were also undertaken at regular intervals during the testing phase.

3.2.4 Pavement Instrumentation

In order to monitor the response of the pavement during the test it was decided to intensively instrument a 10 m-long section of the pavement. For the asphalt surface layer, strain gauge transducers to measure bending strains at the bottom of the layer were specified. Twenty transducers were laid at 0.5 m intervals in each wheel path to measure longitudinal bending strains, with a further twenty transducers in each wheel path to measure transverse bending strains. ARRB Transport Research's H-bar type transducers, which are relatively inexpensive, were used. ARRB staff installed and configured these transducers.

To measure the vertical deflections of each of the lower layers, Bison coil transducers were placed at 8 locations, 1 m apart in each wheel path, each with sensors at the layer interfaces, and at 200 mm into the subgrade. Bison coils have the advantage that they cause minimal disruption to the pavement structure but require considerable care in placing them to achieve accurate alignment. Experienced CAPTIF personnel undertook this placement.

Partial depth deflection gauges were used to monitor deflections and provide a crosscheck on the Bison coils. These are accurate and present fewer difficulties in terms of alignment, but are quite invasive of the pavement structure. For this reason, only three partial depth gauges were placed in each wheel path. These transducers were supplied and installed by ARRB.

Twelve temperature probes were used to monitor temperature, two were to measure air temperature, five at mid-depth in the asphalt layer, and five at the asphalt-basecourse interface.

3.2.5 Preliminary Conditioning

To complete the construction a preliminary conditioning of the pavement consisting of approximately 6000 load cycles uniformly distributed over the area to be trafficked was to be applied. In fact, with these laps and the zero measurements (Section 3.2.6), 20,000 load cycles were completed before the pavement-loading phase commenced.

3.2.6 Zero Measurements

Before the commencement of loading a comprehensive series of zero measurements was made to completely characterise the system and to calibrate the instrumentation. These measurements are described here.

The *longitudinal pavement profile* was measured in the centre of each wheel path and at 0.4 m either side of their centrelines using a DIPStick device. This device produces a reading every 254 mm (10 in.) and resolves to 0.0254 mm (0.001 in.). The circular nature of CAPTIF enables a check on the closure of the profile (i.e. the start and finish point should be identical). This has led to the identification of a small consistent cumulative error. This error has been demonstrated to be linear by measuring the same profile in opposite directions and thus it can be easily corrected for. Measurements in the two wheel paths were repeated to check the measurement accuracy and repeatability.

The *transverse pavement profile* was measured at each of 58 equally spaced stations around the track. The spacing of these stations is 1 m at the mean track radius. Measurements were done using the CAPTIF transverse profilometer. This is a purpose-built device, which consists of a rigid beam mounted radially across the track and an LVDT (linear variable differential transformer) fitted to a small carriage that travels along the beam and measures the distance from the beam to the ground. Readings are taken at 0.025 m intervals and logged with a hand-held computer.

The *instruments*, i.e. in-pavement transducers, Bison coils, strain gauges and partial depth displacement gauges, were monitored while subjected to a range of loading conditions. These conditions consisted of a range of loads with the vehicles operating at test speed, a range of vehicle speeds with the vehicles in test load condition, the dynamic loadings initiated by two artificial bumps at various positions, changes in the transverse position of the vehicles relative to the transducers, and FWD loading.

The test axle load was 10 tonnes, which converts to a wheel load for each SLAVE of 49 kN. To assess the *effects of load*, measurements were made at 45 km/h with vehicle loads of 40 kN, 49 kN and 58 kN.

The nominal test operating speed was 45 km/h. To assess the *effects of speed*, measurements were taken at creep, 42 km/h, 45 km/h and 48 km/h, with the vehicle loaded to 49 kN. The variations about the operating speed were used to assess the significance of speed differences between the inner and outer wheel paths that occur because of the different vehicle radii.

To evaluate the *effects of a SLAVE unit* on the other wheel path, readings were taken from the transducers in the adjacent wheel path as the vehicle passed at crawl speed with a wheel load of 49 kN. The *effect of transverse location* on gauge response was determined by monitoring the H-bars, Bison coils and partial depth gauges in one wheel path as the transverse vehicle position was changed in 50 mm increments.

The *pavement response to induced dynamic loading* was established through the use of two different bumps, one 300 mm long by 25 mm high, and the other 4000 mm by 25 mm high. The short bump was designed to excite the axle hop resonance while the long bump excited the body bounce resonance. In both cases the vehicles were laden to 49 kN and run at 45 km/h. For cross-referencing purposes, similar bumps were used on real trucks on instrumented pavements in Research Element 2 of the DIVINE project, which was conducted in Finland and the USA. It is worth noting that, while these bumps caused little difficulty to the real trucks in Research Element 2, they were quite severe in the CAPTIF environment. This is probably because the wheelbase effects of a complete truck have a moderating influence on the severity of this type of excitation.

The *pavement response to FWD loading* at 50 kN was measured under the H-bars, the partial depth gauges, and the Bison coil arrays.

Basic vehicle parameters were measured. The laden vehicles were weighed using a portable scale. The static tyre deflections and imprints were measured under 31, 40, 49, and 58 kN loads. Similarly the static suspension stiffness was measured for both suspensions. CAPTIF instrumentation provides for calculation of the wheel forces from the measured accelerations of the sprung and unsprung masses. In addition LVDTs were fitted to measure the suspension deflections.

The *vehicle suspensions* were characterised by monitoring this instrumentation on each vehicle while the vehicle was driven at crawl speed over the standard 80 mm high ramp, as defined in EC Directive 90/486/EEC. The suspension deflection signal was analysed to determine the fundamental bounce frequency of the suspension and its damping. This test was repeated with 30 mm and 60 mm bumps to investigate amplitude dependency of this response.

The *wheel forces* as measured by the accelerometers were monitored for each of the test conditions used for the pavement response tests described above. Additionally the effect of speed and pavement profile on dynamic wheel forces was determined by measuring these wheel forces at creep, 10, 20, 40, 42, 45 and 48 km/h with the vehicles loaded to 49 kN. These tests were repeated with the vehicles being run in the opposite wheel paths.

3.2.7 Testing the Pavement

On completing the zero measurements, trafficking of the pavement commenced using the following testing procedure. A pre-determined number of load cycles were applied, then loading was halted, and a series of measurements was taken. This sequence was then repeated. This process was continued until one of the specified failure conditions was achieved. Loading was to be halted if the pavement temperature went outside the range 5°C–35°C. If the temperature fell below the minimum, testing would not recommence until the temperature had been back within tolerance for four hours. If above the maximum, testing would not recommence until the temperature had been back within tolerance for two days.

The number of load cycles (in 000s) between measurements was planned to be 10, 20, 30, 40, 60, 80, 100, 150, 200 and then in increments of 50. The higher frequency initially was to monitor the effects of post-construction compaction. It was anticipated that the measurement interval from 150,000 cycles onwards might be adjusted to fit the observed rates of change. In fact, this planned loading sequence was followed exactly up to the 300,000-load interval, after which the increments were increased to 100,000 load cycles.

The potential number of measurements that could be made at each measurement interval was very large. To keep the time and hence cost involved in taking these measurements and processing the data, a complete data set was collected only every third measurement cycle, and a subset of only the most important data was collected every measurement cycle. These two measurement sets are as follows:

(i) Complete set

For the complete data set, the pavement wear, primary response and structural condition were monitored.

- *Performance measurements*
  The longitudinal profile was measured in the centre of each wheel path and at 0.4 m either side of each wheel path using the DIPstick, the transverse profile was measured at each of the 58 stations around the track using the CAPTIF profilometer, and the cracking was recorded using clear plastic overlays and coloured marker pens. These data were processed to produce IRI values and PSD functions for the longitudinal profiles, maximum rut depth for the transverse profiles, and numerical values for the level of cracking.

- *Structural condition measurements*
  The structural condition of the pavement was measured using both the FWD and the CAPTIF deflectometer at each station in each wheel path.

- *Primary response measurements*
  The pavement response to vehicle loading was measured by recording the strain gauge signals, the deflection gauge signals, and the Bison coil signals for each of the steady speed wheel force measurement conditions specified below, and at crawl speed with minimal dynamic wheel forces.

The vehicles' condition and their interaction with the pavement were measured as follows:

- *Vehicle-suspension condition measurements*
  Each of the vehicles was driven over the EC bump at crawl speed and its response monitored. The bounce frequencies and damping rates calculated from this test provided an indication of changes in the suspension behaviour. Some variation was considered acceptable in that it reflects real fleet conditions and tends to moderate the spatial repeatability caused by using a single suspension.

- *Wheel force measurements*
  The wheel forces generated by each vehicle were measured at the mean operating speed (45 km/h) used in testing. Each vehicle also had its wheel forces measured at this speed while operating in the wheel path normally trafficked by the other vehicle. This provides additional information on the relationship between pavement profile and dynamic response.

(ii) Reduced data set

The reduced data set for intermediate measurement intervals was as follows. A 10-m section of track that includes all the pavement response transducers was intensively monitored.

- *Performance measurements*
  Longitudinal profiles were measured only in the wheel paths. Transverse profiles were measured at every station only in the 10-m section of track, and at every sixth station over the remaining 48 m. Cracking changes were monitored around the complete track.

- *Structural condition measurements*
  None.

- *Primary response measurements*
  The pavement response measurements were only taken for the steady speed wheel force measurement conditions specified below.

- *Vehicle-suspension condition measurements*
  The EC bump tests were done.

- *Wheel force measurements*
  The steady speed wheel force measurements were done at only one speed.

3.2.8 Assessing Failure

Testing was to continue until the pavement in one of the wheel paths reached the specified failure criteria. These were either when the permanent vertical deformation from the original profile exceeded 25 mm, or when the cracking exceeded 5 m/m^2 over 50% of the trafficked area. At this point the load was to be removed from the vehicle in the wheel path being used, and the testing was to continue until the same failure criterion was reached in the other wheel path. At this point the test would be complete. In fact the pavement reached 1.7 million load cycles or four times its design life without the failure criteria being met. If the rate of pavement deterioration that was occurring at that time continued without accelerating, the failure criteria would not have been met until about 3 million load cycles. From both time and cost considerations the test could not be extended further.

3.2.9 Post-mortem Analysis

On completion of the test the pavement was excavated carefully and an analysis of the failure mechanism was undertaken. Six trenches were cut across the full pavement width to measure the layer profiles, and retrieve material samples for laboratory tests. Asphalt core samples were cut for resilient modulus testing in the Materials Testing Apparatus (MATTA) in the University of Canterbury Civil Engineering laboratories. Holes were cut through the layers at both cracked and uncracked stations, and density measurements and Loadman tests were carried out on the unbound basecourse and subgrade layers.

3.3 Test Report

A full loading report for this test has been published previously (Fussell et al. 1996) and the details are not repeated here. This section contains only a brief overview of the loading programme.

On the completion of the pavement conditioning (6,000 laps) and zero measurements, the regular loading programme commenced on 27 October 1994 at which time 15,517 laps had been accumulated. Between this date and 17 August 1995, 1,684,489 loading cycles with 49 kN static wheel load were applied to each wheel path.This corresponds to 98,000 km of travel for each of the SLAVE units. During this period 24 sets of measurement data were recorded resulting in a total of 182 MB of raw data. A detailed log of the loading and measurement cycles is given in the 1996 report (Fussell et al. 1996).

Most of the measurement systems performed very well. There was a minor problem with the axle accelerometer on the SLAVE with steel suspension, for the measurement cycles between 20,000 and 60,000 load cycles. For this suspension, however, the chassis accelerometer alone provides a good estimate of the wheel force particularly on smoother roads so this was not serious. A laser profiler was commissioned in March 1995 in response to some concerns expressed by IR6 in relation to accuracy of the Dipstick profiler. Both systems were used in parallel from 500,000 load cycles onwards. Good correspondence was obtained between the profiles measured using the two devices, and consequently the Dipstick data are used for the analyses in this report. Of the in-pavement systems only the H-bar strain transducers proved unreliable. Forty of these were installed with the expectation that not all of them would survive construction. In fact they all survived construction but began failing as the loads were applied. Some of the loading conditions to which they were subjected were severe. For example, during zero measurement the pavement strain response to FWD loads was recorded, and so the weight was dropped on the pavement directly over the gauges. By 300,000 load cycles all these gauges had failed. The Bison coil gauges, which monitored vertical pavement layer strains, were much more robust and durable. Of the 32 strain channels installed, 26 performed correctly throughout the test. The partial depth gauges and temperature probes all performed with complete reliability.

CAPTIF itself operated throughout the test without any major failures. Some minor repairs and maintenance were necessary but had only minimal impact on the test programme. The most serious problem was the failure of a wheel bearing in the SLAVE unit fitted with steel suspension, which occurred just before 1.5 million load cycles had been completed. This repair necessitated the removal of the suspension assembly. This and the subsequent re-assembly resulted in a reduction in the dynamic loads generated by this vehicle.

3.4 Results

3.4.1 Introduction

The results of the OECD DIVINE experiment as a whole, and this test (Research Element 1) in particular, have been very extensively analysed and reported (OECD 1998, de Pont 1994, 1997, Pidwerbesky 1995, Sharp 1996, Sharp & Moffatt 1996, Steven & de Pont 1996). For detailed results the reader is referred to these works. In this Section 3.4 only a summary of the more significant results is presented.

3.4.2 Pavement Construction

Although the design specified an 80 mm asphalt layer, the pavement as constructed had a mean asphalt layer thickness of approximately 88 mm. However, this thickness was relatively uniform and so the criterion of minimising the transverse variability was satisfied. The basecourse layer thickness was very close to the specified value and the variability of this layer was well below the specified tolerance, while the subgrade layer had a CBR value at the upper end of the specified range.

The principal objectives of the pavement construction were to match the design specifications as closely as possible and to minimise the transverse variability in the pavement. The two wheel paths were required to be as identical as possible. Relatively tight tolerances were specified and extensive quality control testing was undertaken to ensure that they were met. Table 3.1 shows a summary of the thickness measurement results.

Table 3.1 Summary of pavement layer thicknesses (mm).

Layer	Basecourse	Asphaltic Concrete
Mean layer thickness (mm)		
Inner wheel path	197.6	87.2
Outer wheel path	198.6	87.9
Standard deviation of layer thickness (mm)		
Inner wheel path	6.3	8.6
Outer wheel path	9.5	6.8
Differences in layer thicknesses (mm)		
Mean difference between wheel paths	1	0.7
Standard deviation of differences	5.4	3.9

Consistent subgrade densities were recorded with the standard deviation less than 2% of the mean. The laboratory resilient modulus of the subgrade material was 105 MPa (at a moisture content of 10%, confining pressure of 13.8 kPa, and deviator stress of 27.6 kPa). Similar uniformity was achieved with the crushed rock basecourse layer. The well graded aggregate had a maximum particle size of 20 mm and a minimum of 80% crushed faces. The laboratory resilient modulus of the basecourse aggregate is 280 MPa (at a moisture content of 2.5%, confining pressure of 103 kPa, and deviator stress of 207 kPa). The Loadman (a single-point falling weight deflectometer) in-situ test on the basecourse gave a modulus of 250 MPa.

The bulk density of the asphaltic concrete was 2.50 t/m^3, the air voids were 2.0 % and the Voids in the Mineral Aggregate (VMA) was 14.6 %. Twelve cylindrical AC cores were extracted from the pavement and tested for resilient modulus at different temperatures. The measured resilient moduli at 10°C and 25°C were 4600 and 1400 MPa respectively.

3.4.3 Zero Measurements

After the pavement construction and conditioning laps had been completed, a series of comprehensive "zero" measurements was made to characterise the pavement –vehicle interaction systems and to calibrate the instrumentation. These have been described in some detail in other reports (de Pont 1994, de Pont & Pidwerbesky 1995). By the time these were completed 20,000 load cycles had been applied to the pavement.

Structural capacity measurements using the CAPTIF deflectometer and the FWD showed a low level of transverse variability, a greater but still low level of longitudinal variability, and no discernible difference between the two wheel paths. Figure 3.1 shows the peak deflections under FWD loading at each station in each wheel path.

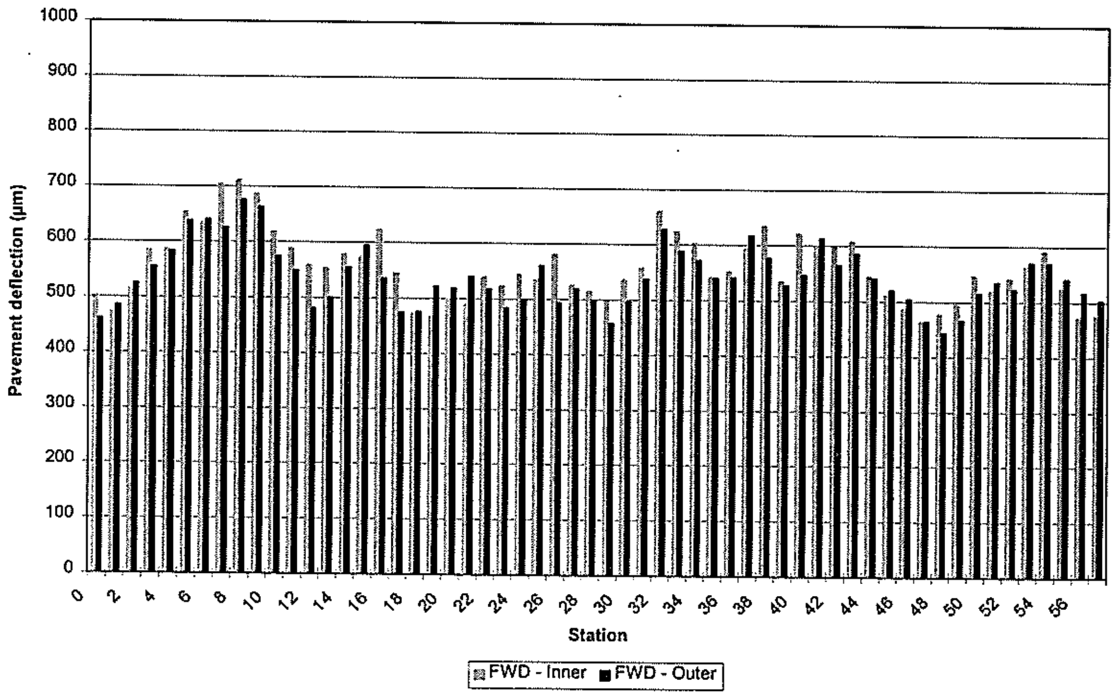


Figure 3.1 Comparison of peak pavement deflections under 50 kN FWD loading for inner and outer wheel paths.

As stated in Section 2.2, one vehicle was fitted with an air spring suspension with a hydraulic shock absorber, and the other had a multi-leaf steel spring. The suspensions had substantially different dynamic wheel force characteristics, with the air suspension generating dynamic loads that were substantially less than half those generated by the steel spring.

At every measurement interval the suspension response was characterised using the EC "equivalent-to-air" bump test. The suspension deflection response at the start of the test was that shown in Figure 3.2. This shows quite clearly that the air suspension has a significantly lower natural frequency than the steel. The damping of the steel suspension, however, is just as good as that on the air suspension. The reason for this is that the magnitude of the 80 mm drop is sufficient to overcome the stiction between the leaves on the steel suspension, and consequently the friction damping is quite effective. With lesser excitations the steel suspension remains "locked" and most of the springing comes from the tyres which have very low damping.

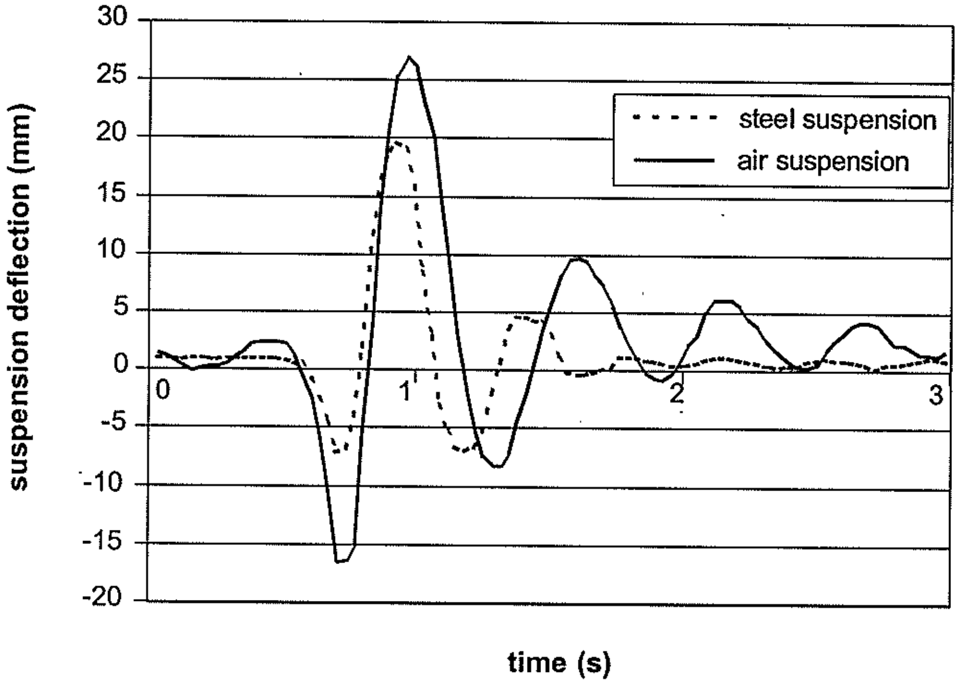


Figure 3.2 Response of the two suspension systems to 80 mm EC bump test.

3.4.4 Loading Measurements

The pavement started to show signs of deterioration as early as 60,000 loading cycles, with the rapid development of a rut in the outer wheel path from stations 18 to 23. This occurred during a period of unusually hot weather when the temperature of the asphaltic concrete reached 34°C. The rate of deterioration slowed down after 10^5 loading cycles and the localised rut failed to generate any significant vehicle response at locations further around the track. Analysis of the measurements taken during construction showed lower than average values around the area of the rut, but they were still within the specified values. After the rate of growth of the rut had slowed down, it became apparent that the failure criteria were not going to be attained before 500,000 loading cycles were applied.

After the completion of 250,000 loading cycles, all the H-bar gauges for measuring horizontal tensile strains in the asphalt had failed, because the strains measured in the asphaltic concrete were greater than the capacity of the gauges. Between 100,000 and 800,000 loading cycles, the pavement condition deteriorated slowly in both wheel paths. After 840,000 loading cycles, the pavement started cracking in both wheel paths. The initial cracks were longitudinal in direction but migrated to block cracking as transverse cracks appeared and linked to the longitudinal cracks. The cracking progressed rapidly until 1,300,000 loading cycles when the rate of increase slowed down markedly. Loading continued until 1,700,000 loading cycles had been completed and then the test was terminated without reaching either of the failure criteria, because of budget and time constraints. However enough useful data had been collected, and the two suspension types had exhibited different behaviours, to enable some conclusions to be drawn from the experiment.

All the basic pavement responses measured showed the same trends over the duration of the test: the rate of change was high during the first 100,000 loading cycles, after which the rate of change was much lower. In a typical project at CAPTIF, the first 40,000 loading cycles show high rates of pavement change as a result of post-construction consolidation.

Figure 3.3 shows the measured vertical compressive strain response of the basecourse, and upper and lower subgrade with respect to applied loads. The values increased in the first 100,000 loading cycles only and then remained relatively constant. Most of the difference in strain values for readings at the same depth is attributed to the varying dynamic wheel forces because the variation in the pavement and subgrade properties between stations was very low.

The pavement deflection as measured by the CAPTIF Deflectometer followed the same trends as the other types of data: a rapid increase occurred in the first 100,000 loading cycles, and then the values remained reasonably constant.

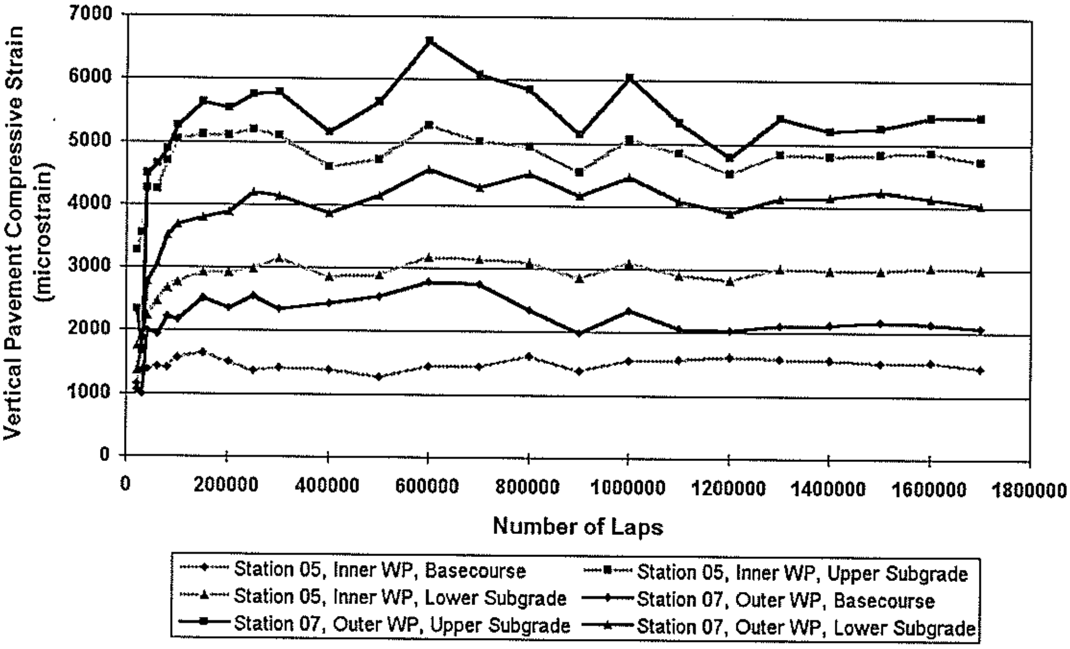


Figure 3.3 Vertical compressive strains of the pavement v number of loading cycles.

Three measures of the pavement surface condition were monitored regularly during the test to provide indications of wear. These were: the transverse profiles, which can be analysed to determine the pavement rutting, the longitudinal profiles which can be analysed to calculate various unevenness and roughness measures, and the pavement cracking.

The maximum rut depth in the outer wheel path (under the vehicle carrying the multi-leaf steel suspension) was 37 mm, with an average value over the 58 stations of 11 mm. In the inner wheel path, which was loaded by the vehicle fitted with the air suspension, the maximum rut depth was 12 mm, with an average value of 9 mm. If the region which contains the large rut in the outer wheel path (stations 18 to 23) is removed from the calculations, the outer wheel path has a maximum rut depth of 14 mm and a mean of 8 mm, while the values for the inner wheel path are unchanged. This indicates that apart from this rut, the level of rutting in the two wheel paths was similar.

The longitudinal profiles can be used to calculate the International Roughness Index (IRI). At the commencement of the test the IRI values of the inner and outer wheel paths were 4.8 mm/m and 4.1 mm/m, respectively. However, this apparent difference was offset by the speed differences due to the difference in radius in the two wheel paths. At the same nominal speed, each vehicle generated virtually identical levels of dynamic loading in the two wheel paths. As the test proceeded the pavement became rougher. The changes in IRI values for the two wheel paths are shown in Figure 3.4. As can be seen, the initial trend is for a small reduction in IRI or smoothing of the pavement. The initial levels of roughness are relatively high for what was a very uniform and carefully constructed pavement. The reason may have been the hand screeding method of pavement construction which resulted in higher amplitudes for the short wavelength components of pavement unevenness. Because the screeding was done radially this effect would be greater on the inner wheel path where the corresponding wavelengths would be shorter than on the outer. However, this wavelength difference would exactly be matched by the speed difference in the two wheel paths. This hypothesis is supported by the fact that each of the two suspensions produced identical levels of dynamic loading in both wheel paths at the start of the test. From 200,000 load cycles onwards, the IRI indicated a gradual slow increase in roughness with relatively little difference in the two wheel paths.

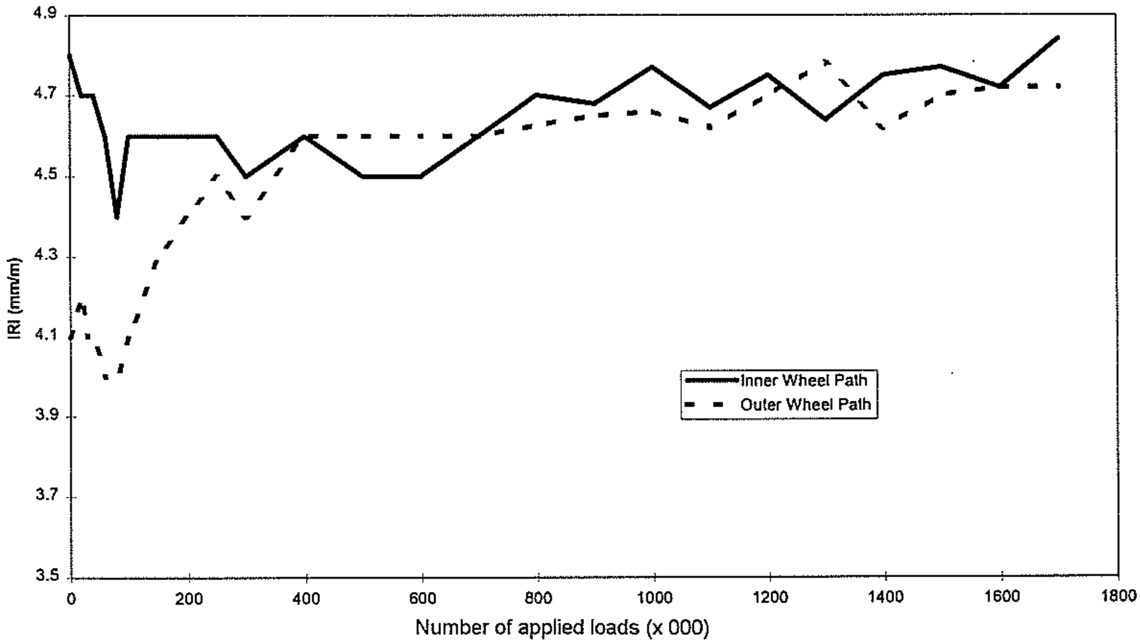


Figure 3.4 Changes in IRI with load cycles for inner and outer wheel paths.

It should be noted that in the CAPTIF environment, IRI has some limitations. IRI is calculated from the response of a theoretical quarter car model to the pavement profile. This model "travels" at 80 km/h and is influenced primarily by pavement wavelengths between 1 m and 40 m. It is usually recommended that the IRI is calculated on sections at least 200 m long in order to obtain a reasonable sample of the longer wavelength components. At CAPTIF the track is only 58 m long and so the sample length is necessarily shorter than desirable. Moreover, the SLAVE units at CAPTIF have a maximum operating speed of 50 km/h and so respond to a different range of wavelengths to that indicated by IRI. While it would be possible to use an alternative roughness measure to IRI (using a lower vehicle speed), this would not relate back to measures used on in-service pavements.

3.4.5 Dynamic Wheel Forces

In many respects the dynamic wheel forces, which were measured for both vehicles in both wheel paths at each measurement interval, provide a better picture of the increasing roughness. These wheel forces are shown in Figure 3.5. Both suspensions indicate that the apparent roughnesses of the two wheel paths were identical at the start of the test. In the early part of the test the outer wheel path increased in roughness more rapidly than the inner. From 200,000 load cycles on, both wheel paths become steadily rougher. The rate of increase was higher on the outer wheel path. The sudden reduction in dynamic loads from the steel suspension SLAVE at 1,500,000 load cycles occurred because a wheel bearing failure necessitated the disassembly of this vehicle. Although no suspension maintenance was carried out, the disassembly and re-assembly of the suspension improved its performance. By the end of the test this SLAVE was tending back towards its previous levels of dynamic load.

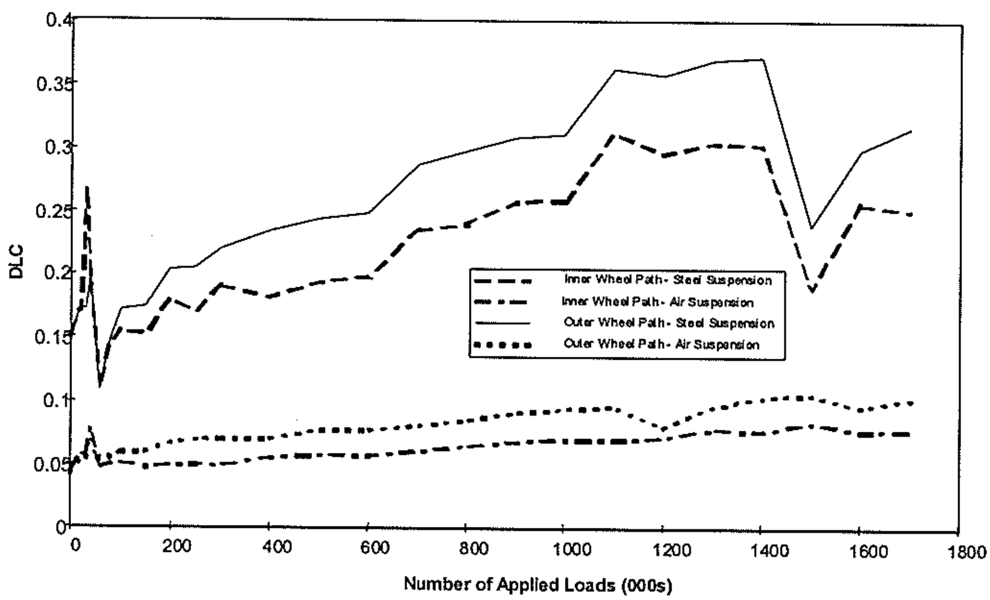


Figure 3.5 Dynamic wheel forces (DLC) v number of load cycles for inner and outer wheel paths.

To investigate possible relationships between the applied loads and the profile changes, cross-correlation functions were calculated. An explanation of what these cross-correlation functions represent is as follows. If we have two quantities which vary around the track, then calculating the correlation coefficient between these two variables gives a measure of how closely related the two quantities are to each other. A correlation coefficient of unity indicates that the two quantities are completely related, while a coefficient of negative unity also indicates that they are completely related but that when one increases the other decreases and vice versa. A correlation coefficient of zero indicates that the quantities are unrelated. However, it is possible that the two quantities are related but separated by a delay or phase shift. Simply calculating the correlation coefficient will not show this relationship. However, if the correlation coefficient for each possible phase shift is calculated, a function called the cross-correlation function is obtained. By plotting this cross-correlation we not only find if the quantities are correlated but also at what phase shift the maximum correlation occurs.

Figures 3.6 and 3.7 show the cross-correlation functions between the loads at 1,000,000 load cycles and the profile changes from 400,000 load cycles till the end of the test at 1,700,000 load cycles, for the inner and outer wheel paths respectively.

The load interval for the profile changes was chosen to avoid undue influence from the rut, which formed early in the test. In both wheel paths the maximum correlation (in absolute terms) is negative. That is, high wheel forces correlate with low profile points and vice versa. The peak correlation values are relatively low, –0.27 for the inner wheel path and –0.46 for the outer, but this outer value is significantly higher than the inner. Both functions are periodic with the inner wheel path having a wavelength which, at the test speed, corresponds to a frequency of approximately 1.5 Hz and the outer wheel path having one that corresponds to 2.15 Hz. In both cases these match well with the natural frequencies of the suspensions applying the loads. Thus it appears that dynamic loads influence the change in profile, but that this is not the only influence. For the outer wheel path and the steel suspension this effect is significantly stronger, in that the correlation coefficient values are higher and the periodicity is much clearer.

3.4.6 Pavement Cracking

The cracking in the asphaltic concrete helped provide a key indicator as to the difference in performance of the two suspension types. Cracking occurred at 52 of the 58 stations in the inner wheel path and the total crack length was 64.1 linear metres. In the outer wheel path, 48 stations had cracking, but total crack length was 87.0 linear metres. The failure criterion was exceeded at five and thirteen different stations in the inner and outer wheel paths, respectively. The cracking that was measured between stations 7 and 8 is illustrated in Figure 3.8.

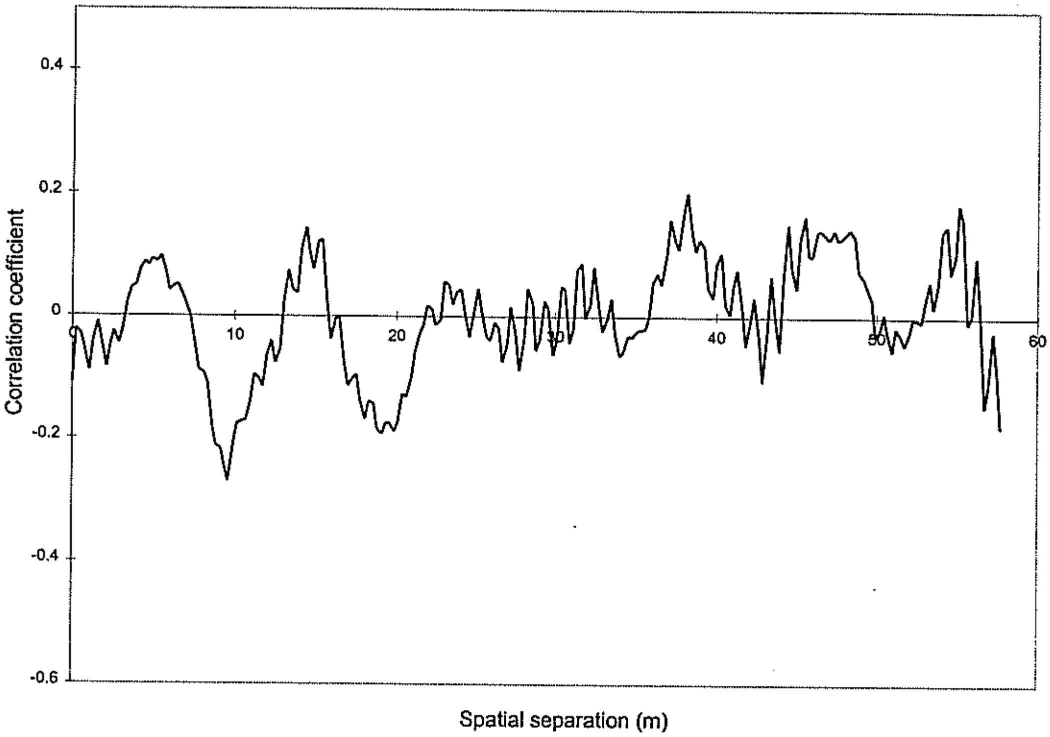


Figure 3.6 **Cross-correlation between wheel forces at 1,000,000 load cycles and profile change between 400,000 and 1,700,000 load cycles, for inner wheel path.**

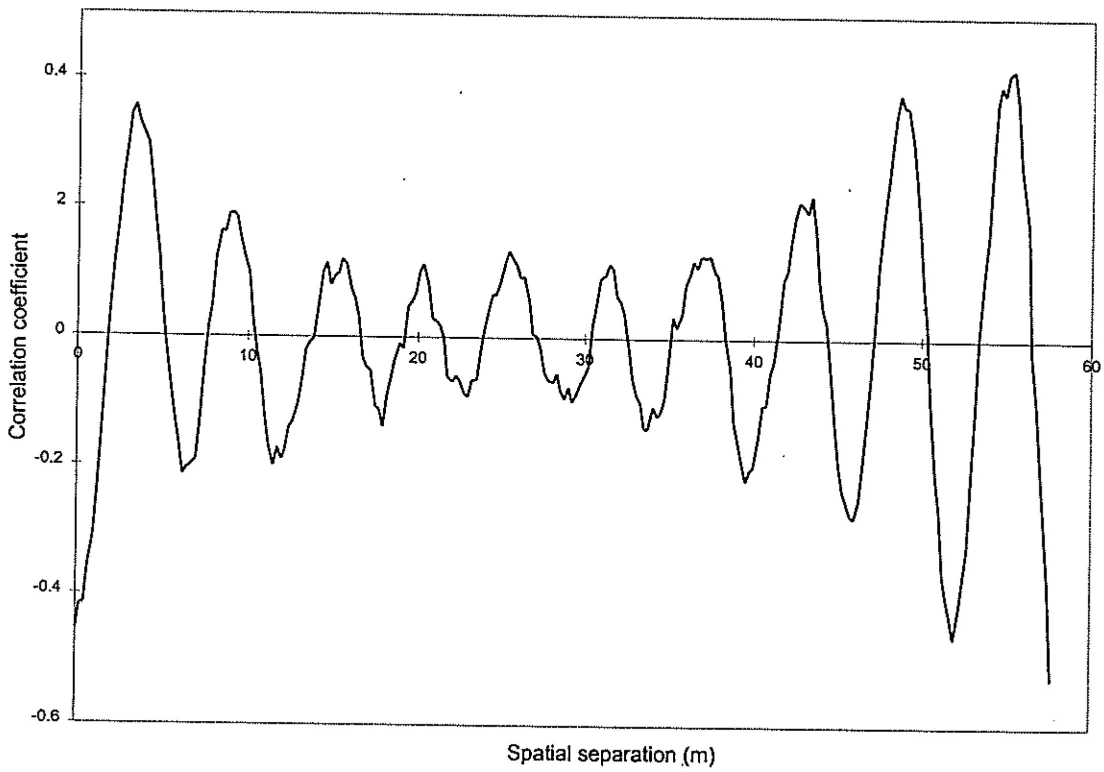


Figure 3.7 **Cross-correlation between wheel forces at 1,000,000 load cycles and profile change between 400,000 and 1,700,000 load cycles, for outer wheel path.**

The cracking in the inner wheel path under the air suspension was uniform in severity around 90% of the track, while the cracking under the steel suspension in the outer wheel path was quite severe in places and quite low in other places. Correlation is relatively good between the cracking and the pavement deflection in the outer wheel path, with a correlation coefficient of 0.82, whereas in the inner wheel path the correlation coefficient is 0.54. Similarly relatively good correlation in the outer wheel path between the rut depth and cracking is obtained (coefficient of correlation is 0.81), while that of the inner wheel path is only 0.32.

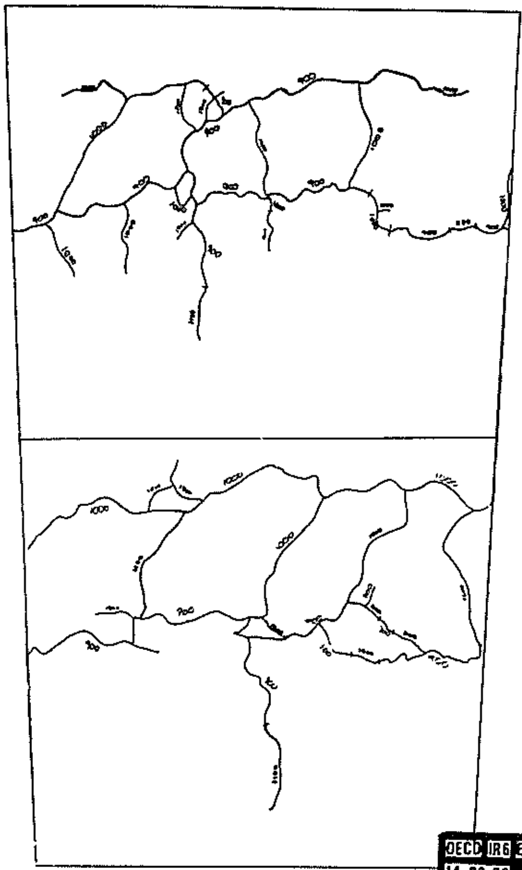


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Figure 3.8 Cracking patterns at stations 7 and 8.

The difference in the levels of distress between the two wheel paths is not as high as some of the models would predict. However, because the rates of increase in distress are low, even a small difference in these rates can result in a substantial difference in pavement life (that is, the number of loads needed to reach some failure criteria). In all three measures, rutting, cracking and roughness, the wheel path that was loaded by the steel suspension suffered higher levels of distress than the wheel path that was loaded by the air suspension.

There is a strong correlation between the different types of distress in the outer wheel path and a poor correlation in the inner wheel path. This supports the hypothesis that dynamic loadings contribute to pavement wear. The steel multi-leaf suspension had much higher dynamic loads and so the total load (static + dynamic) was much more periodic and resulted in periodic patterns of damage.

The dynamic loads of the air suspension were much lower and so the total load is dominated by the static component. Hence the resulting damage patterns are much more evenly distributed.

3.4.7 Post-mortem Excavations and Analysis

Following the completion of the loading, the post-mortem included cutting cores from the asphaltic concrete for resilient modulus testing in the Materials Testing Apparatus (MATTA) in the University of Canterbury Civil Engineering laboratories.

Six transverse trenches (two at stations with maximum deformation, two at stations with minimum deformation, and two at stations with average deformation for each wheel path) were excavated through the asphalt and basecourse layers. These were to measure the layer profiles, and to retrieve material samples for laboratory tests. Small holes (three at uncracked stations and three at cracked stations, in each wheel path) were cut through the layers to measure densities and carry out Loadman tests on the unbound basecourse and subgrade layers.

The excavations revealed that most of the vertical deformation had occurred in the unbound granular layer. The average maximum compaction within the asphaltic concrete in both the inner and outer wheel paths was 4 mm; and there was negligible compaction in the subgrade under both types of suspensions. The average maximum compaction within the unbound granular basecourse layer in the inner (air bag suspension) and outer (steel suspension) were 7 mm and 11 mm, respectively. The basecourse aggregate properties were the same in each wheel path, as shown in Table 3.2.

The asphalt moduli tests in the MATTA showed that the asphalt moduli in both the cracked and uncracked sections were similar, and had not changed since the start of the loading routine.

Table 3.2 Post-mortem properties of the unbound basecourse aggregate.

Wheel path	mc (%)	9.5mm	4.75mm	psd	Sample Size		Loadman Modulus (MPa)		
		Broken Faces (%)		(n)	No.	No.	Before	After	No.
Averages									
Inner	1.6	100	84	0.49	11	58	135	80	5
Outer	1.8	99	84	0.49	12	58	134	82	5
Standard Deviations									
Inner	0.3	0.8	3.1	0.03	11	58	17	6	5
Outer	0.4	0.8	2.7	0.03	12	58	14	10	5

mc - moisture content (%) psd - particle size distribution (n)

3.5 Summary and Conclusions

This pavement test was undertaken as Research Element 1 of the OECD DIVINE project. As a consequence a substantial analysis effort, involving not only the authors of this report but researchers from a number of other OECD countries, was undertaken. This work has been written up in a number of published research papers and reports, which have been referenced at the start of this chapter. Rather than replicate the content of these other publications, only some highlights of the findings have been presented. However, in this Section 3.5 the findings of the main research report (Kenis & Wang 1998) covering this test are summarised.

DIVINE Research Element 1 investigated the effects of the dynamic loading generated by two different suspensions, an air spring with viscous damper and a multi-leaf steel spring, on the performance of a full-scale flexible pavement. The investigation focussed on:

- the changes in layer thickness and material properties subjected to accelerated loading,
- primary pavement response,
- the rates of damage progression subjected to accelerated loadings, and
- the effect of initial pavement structural variability and dynamic loading on pavement performance.

The results showed the following points.

- Although there was an unexpectedly relatively high negative correlation between the constructed thickness of the AC layer and base layer of the pavement, this probably occurred because of the efforts to make the pavement surface flat and smooth.

- Based on FWD surface deflections at construction, the two wheel paths showed little difference in structural integrity. However, an increase in the FWD surface deflections around station 21 (between stations 18 and 24) on the outer wheel path just before trafficking at 20,000 load repetitions, is an indication that the pavement had weakened in that area. As a result a rut was generated on the outer wheel path (trafficked by the steel spring suspension) between stations 18 and 24 during the initial stages of trafficking. Because of the high FWD deflections at 20,000 load cycles, the rut in that area is believed to be related to the combined effect of dynamic loading and the weakened pavement in that area.

- Because of the occurrence of the rut between stations 18 and 24, two sets of data – Select data (data set not containing the rut) and All data (data set containing the rut) – were used in the analysis. In most cases Select data were used for the outside wheel path analyses, whereas for the inner wheel path both All data and Select data were used since the inner wheel path contained no obvious localised weakened area.

- The pavement materials appear to have changed only slightly over the life of the pavement. There was a modest increase in the modulus of the asphalt concrete.

  Consolidation of the basecourse layer combined with high dynamic loading led to the rut that occurred around station 21. Overall the basecourse layer experienced the greatest consolidation. However, no significant difference was obvious in the before and after gradation tests.

  The high levels of rutting in the outer wheel path around station 21 were not caused by excessive moisture in the basecourse. The subgrade failed to show any significant signs of rutting (except in the region around station 21 on the outer track) and the subgrade properties remained constant throughout the project.

  Densification of the asphalt concrete and basecourse layers was mainly responsible for overall surface rutting that was observed.

- The cross-correlation analysis conducted for pavement primary responses showed some good correlations for vertical layer deformation, dynamic loading, and initial primary responses in the pavement and subgrade for the inner wheel path.

- The outer wheel path was subjected to significantly greater dynamic loads and had a larger increase in roughness than the inner. This was the case even after the rut had stabilised.

- Various statistical measures of the wheel path profiles with load repetitions were used to assess profile unevenness (roughness). In general those based on the PSD of the profiles provided the most insight, though the wheel force response of the test vehicles was also a useful indicator.

- Spectral analysis of the profile changes showed a match between the wavelengths of the changes and the natural frequencies of the vehicle suspensions, particularly on the outer wheel path indicating that dynamic wheel forces do contribute to profile changes.

- The IRI was shown to have some limitations as a measure of roughness in the CAPTIF environment where the track length is relatively short and vehicle speeds are significantly lower than the IRI standard speed of 80 km/h.

- The change in mean vertical surface deformation and mean rut depth on both wheel paths was similar with the use of Select data, confirming theoretical assumptions that dynamic loading had no effect on the accumulation of mean surface deformation.

- Since the coefficient of variation is a measurement of the fluctuation of values of a variable around its mean, the variations of surface deformation, rut depth, and profile give an indication of the fluctuation in surface elevation around its mean along the pavement. This fluctuation can also be seen as an indicator of road roughness. Based on this argument, it was observed that, at the end of testing, the coefficient of variation of the surface deformation on the outer wheel path was about 28% greater than that on the inner wheel path if using Select data for both wheel paths. Similarly at the end of testing, the coefficient of variation of rut depth on the outer wheel path was 19% greater than that on the inner if using Select data for both wheel paths. As the coefficient of variation of the FWD deflections was only 4% higher on the outer wheel path, this indicates that the steel spring suspension did contribute to a significantly higher variability in the surface profile on the outer wheel path.

- Based on cross-correlation analysis, the initial profile had little influence on the final profile in the two different tracks regardless of the type of suspension.

- Cross-correlation functions at different load repetitions between pairs of variables such as FWD surface deflection, profile elevation, surface deformation, rut depth, cracking, and wheel force, were calculated to determine the strength of their relationships.

  The coefficient of correlation between surface deformation and FWD surface deflection for the inner wheel path was about 0.75 while for the outer wheel path it was about 0.30. The stronger correlation on the inner track indicates that pavement variability played a more dominant role in explaining the pattern of surface deformation on the inner track. The poorer correlation on the outer track indicates that factors other than pavement variability, such as load, played a bigger role.

  The coefficients of correlation between rut depth and FWD surface deflection for both wheel paths were similar at around 0.6.

  The coefficient of correlation between wheel force and surface deformation normalised by FWD surface deflection was about –0.2 for the inner wheel path and about 0.4 for the outer path. The stronger correlation for the outer wheel path indicates that wheel force plays a stronger role in explaining surface deformation on the outer wheel path.

- The cracks that occurred in the asphaltic concrete mostly originated from the surface and were not full-depth cracks. The cracks that did occur also provided an indicator of the difference in performance of the two suspension types as follows:

- Cracking in the outer wheel path (steel suspension) was 36% greater than that in the inner wheel path (air suspension). The cracking here is the total measured length of cracks around the wheel path. This cracking was more localised in the outer wheel path, with slightly fewer cracked stations but with more than twice as many stations reaching the cracking failure criterion. In the outer wheel path cracking was strongly related to profile changes and to rut depth, while the relationships in the inner wheel path were much weaker. However, if using Select data, the differences in cracking performance between the two paths were insignificant.

- Cross-correlation between surface cracking and rutting yielded coefficients of correlation of 0.8 for the outer and 0.4 for the inner wheel path. Given that the mean rut depths of the two tracks were similar, the greater variation of rut depth on the outer wheel path indicates the existence of more locations with high levels of rutting around the path. The higher correlation between cracking and rutting for the outer wheel path indicates that surface cracking started when rutting had reached a certain level of severity. Therefore, it may be concluded that the steel suspension-induced wheel force caused more localised rutting damage and more localised cracking.

4. THE THIRD PAVEMENT TEST

4.1 Introduction

The primary objective of this study is to establish the relationship between dynamic wheel forces and pavement construction used for the New Zealand roading network. Two secondary objectives are: to determine the relationship between applied wheel loads and pavement strains for typical New Zealand pavements; and to validate the EC "equivalent-to-air" test for heavy vehicle suspensions as a measure of "road friendliness" suitable for use in New Zealand.

4.2 Test Configuration

For this test a typical New Zealand pavement design consisting of a 25 mm asphalt layer, simulating a chipseal surface laid over a 250 mm crushed rock basecourse and a silty clay subgrade of CBR 10-12 was used. This is basically the same design as the pavement used in the first test in this research (Section 2 of this report). Using the NRB design charts this pavement has a design life of 350,000 ESA. The wheel load for this test was again 49 kN, which is equivalent to a 10 tonne axle. As the reference axle load for wide-based single tyres in New Zealand is 7.2 tonnes, then, based on the fourth power relationship, each load cycle applies 3.72 ESA. On this basis the pavement design life is approximately 94,000 load cycles.

However, the issue of pavement design life is complicated. The discussions so far have been based largely on the values predicted by the NRB design charts (NRB 1989), which were the basis of New Zealand pavement design practice until 1995 when the AUSTROADS procedures (AUSTROADS 1992) were adopted by Transit New Zealand. AUSTROADS uses ESA (equivalent standard axles) rather than EDA (equivalent design axles) as used by NRB, but these are similar in magnitude. More significantly the relationship between subgrade strain and pavement life is quite different with the AUSTROADS guide generally predicting a longer life.

An analysis of the above pavement structure was undertaken using the AUSTROADS Pavement Design and Rehabilitation Guide (1992) and CIRCLY, a computer program that uses linear elastic theory to predict the pavement response (Wardle 1996). The input parameters are given in Table 4.1. Suitable values for the moduli of the different layers were based on the assumptions listed in the design guide. The resultant strain from CIRCLY at the top of the subgrade was 1420 microstrain. This value is then used as an input to the AUSTROADS Subgrade Strain criterion, which is:

$$N = \left[\frac{8511}{\mu\epsilon}\right]^{7.14}$$

where N is the allowable number of repetitions of the axle load, and
$\mu\epsilon$ is the vertical compressive strain at the top of the subgrade (in units of microstrain)

When the allowable number of axle loads is determined in accordance with the calculated pavement strain and the strain criteria, the number of design repetitions is 357,000. Thus, the design life of the pavement in accordance with accepted AUSTROADS procedures is 357,000 passes of a wide-based single tyre loaded to 49 kN, that is, nearly four times as many as the 94,000 passes given by the NRB design guide.

Table 4.1 Input parameters for CIRCLY model of pavement structure.

	Layer		
	Asphalt Concrete	**Basecourse**	**Subgrade**
Thickness (mm)	25	250	1200
Vertical Modulus (MPa)	2800	500	10xCBR =10x11=110
Poisson's ratio	0.2	0.35	0.35
Automatic Sublayering	No	Yes	No
Anisotropic material properties	No	Yes	Yes
Degree of anisotrophy	1	2	2
Wheel Load (kN)	49		
Contact pressure (kPa)	700		
No. of wheel loads	1		

Of the suspensions available at CAPTIF, the air suspension and the multi-leaf steel suspension represent the best and worst performing in terms of road-friendliness, and both were used for the DIVINE test. These two were used again for this third test. Both SLAVE units were loaded to give a 10 tonne axle load and fitted with wide-based single tyres to provide a direct comparison with the results of the second test. As the pavement for the second test (DIVINE project) was much more typical of the flexible asphaltic concrete structures that have been the subject of much of the research on flexible pavements, being able to directly compare the third test results for a thin-surfaced structure will provide valuable insights. As with the previous tests the tyre pressures were set to 700 kPa, and the lateral wander followed a Gaussian distribution with a standard deviation of 50 mm but restricted to ±2 standard deviations.

A short section of the test pavement was instrumented with eight sets of Bison coil transducers at 1 m spacing in each wheel path to measure the vertical compressive strains in the basecourse and subgrade layers. The asphalt bending strains were not measured as the contribution of the asphalt layer to the pavement strength is assumed to be negligible. Air and asphalt temperature probes were placed at 12 locations around the track.

4.3 Test Report

This test was conducted in the period from January to July 1998. Between the completion of the second test and the commencement of this test, the CAPTIF underwent a major maintenance, development and upgrade programme. Some of the modifications were necessary to improve the operational safety of the facility in line with the requirements of the Occupational Health and Safety Act, while others were needed to replace outdated and aging electronic systems for control and measurements. Commissioning and testing of this new equipment was done during the setting up of this test with the result that the total elapsed time for this test was considerably longer than would normally be the case.

In January 1998, the new SLAVE control systems were commissioned, and a new remote data acquisition system was installed. The SLAVE units were then serviced and removed from the track so that the previous pavement could be excavated.

This test therefore effectively started in February with the construction of the test pavement. The SLAVE units were re-mounted on the track early in March, and pavement structural capacity and transverse profile measurements commenced in the week beginning 9 March. In the following week the pavement conditioning laps were applied and the calibration of the measurement systems began. All the other zero measurement tasks were completed by 9 May.

Trafficking commenced in mid-May. By 24 June 150,000 load cycles (exceeding the design life of the pavement) had been completed and, although showing some wear, the pavement was still a long way from failure. It was agreed to extend the test for a further three weeks to allow the pavement deterioration to get closer to the failure criteria. The test was stopped at 300,000 load cycles and the post-mortem testing was completed by 23 July.

4.4 Results

4.4.1 Introduction

As outlined in Section 4.2 the pavement had a design life of 350,000 ESA based on the original design method. With wide-based single tyres and a 49 kN wheel load this corresponds to approximately 94,000 load cycles. The actual test ran for 300,000 load cycles, by which time there was significant pavement wear although the level of rutting was still below the pre-set failure level. Thus the pavement had withstood more than double its design load cycles, and would very probably have survived well past triple that number of cycles. However, using the AUSTROADS design calculations, the design life is 357,000 load cycles, which is very much in line with what happened.

4.4.2 Construction

As with the previous tests the pavement was extensively monitored and tested during construction. Table 4.2 gives a summary of the as-constructed layer thickness values and their variability. As with the DIVINE pavement a good degree of uniformity was achieved.

The compacted density of the subgrade was reasonably consistent, with a mean value of 1668 kg m^{-3}. The standard deviation of the values was 4.7% of the mean value. The average moisture content of the subgrade material was 11.2%, with a standard deviation of 1.0%. The target density for the subgrade material was 1780 kg m^{-3} for a moisture content of 11%. The average Loadman Elastic Modulus values (shown in Table 4.3) measured on the top of the subgrade and basecourse were 74 MPa and 132 MPa respectively.

No specific tests were performed on the asphaltic concrete surfacing, as this layer was assumed to contribute no structural strength to the pavement structure.

Table 4.2 Summary of pavement layer thicknesses (mm) used in the third test.

Layer	Basecourse	Asphaltic Concrete
Mean layer thickness (mm)		
Inner wheel path	252.7	26.9
Outer wheel path	253.0	27.8
Standard deviation of layer thickness (mm)		
Inner wheel path	6.3	2.8
Outer wheel path	7.7	2.6
Differences in layer thicknesses (mm)		
Mean difference	0.3	0.9
Standard deviation of differences	5.7	2.8

Table 4.3 Loadman Elastic Modulus values for subgrade and basecourse of test pavement.

Layer	Lift number	Elastic Modulus (MPa)				Number of measurements
		Average	Standard deviation	Minimum	Maximum	
Subgrade	1	44	16	20	71	36
	2	47	8	36	69	36
	3	46	6	36	56	36
	4	49	6	38	65	36
	5	48	7	35	67	36
	6	55	8	42	85	36
	7	74	12	54	117	36
Basecourse	8	82	9	70	106	36
	9	132	10	106	153	36

4.4.3 Dynamic Wheel Forces

As with the previous tests, the EC bump test was used to characterise the suspensions and to monitor their performance. Figure 4.1 shows the suspension deflection responses for the two suspensions at the commencement of the test. If these are compared to the responses of the same suspension configurations when used for the second pavement test (Figure 3.2), it is readily seen that the suspensions used for these two tests are very similar in performance. Calculating the natural frequency and damping from the bump test gives a natural frequency of 1.93 Hz and a damping of 23% for the steel suspension, and 1.44 Hz and 16% for the air suspension. The values for the steel suspension are very similar to those measured during the second pavement test, while the air suspension has the same frequency but a little less damping. During the second test its damping was 20%.

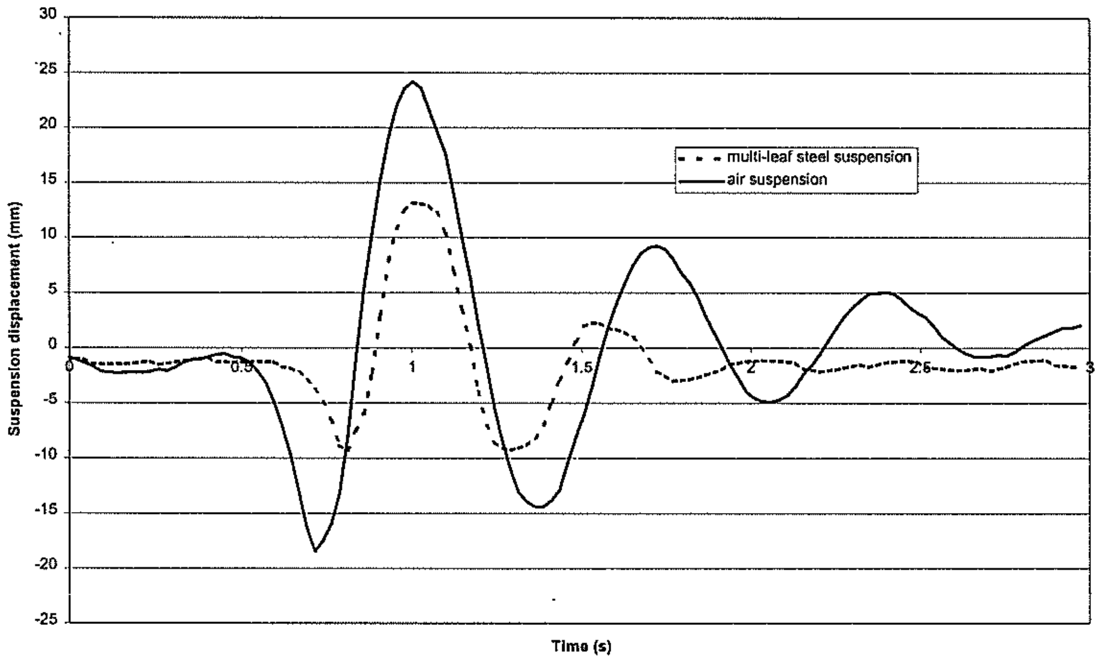

Figure 4.1 Suspension deflection responses to EC bump test for the two types of suspension.

As with the previous tests the dynamic wheel forces generated by the two suspension configurations were measured at various speeds during the zero measurement phase and at testing speed (45 km/h), at each measurement interval. Somewhat surprisingly the difference in the DLC values generated by the two suspension configurations was significantly less than in the second test. The dynamic loads generated by the air suspension at the start of the test were a little higher than in the second test (DLC = 0.055, Figure 3.5), but this is not surprising as the initial pavement roughness was also higher (Figures 3.4 and 4.10). However, the dynamic loads generated by the multi-leaf steel suspension were very much lower than those in the previous test (DLC = 0.086 (Figure 4.3) in spite of the rougher pavement. This is surprising as the EC bump test indicates very similar performances in both instances. A possible explanation may be that CAPTIF underwent a major refurbishment between the two tests, when the suspensions were disassembled and overhauled. This process may have reduced the static interleaf friction so that the spring was able to function at low excitations. During the second test it appeared as if this spring was effectively locked at low excitation levels, and that the main spring element was the tyre, which is stiffer than the spring and has very low damping.

Figure 4.2 shows a comparison between the steady speed (45 km/h) wheel force spectra of the steel suspension between these two tests. This shows that in the third test the largest dynamic response occurs at approximately 2 Hz, which is the natural frequency of the body bounce mode as determined by the bump test. The wheel force spectrum from the second pavement test shows the main response centred at 2.6 Hz, which corresponds to the natural frequency calculated from the whole vehicle mass and the measured tyre stiffness. Interestingly, when this suspension had to be removed and refitted during a wheel bearing repair during the second test, there was also a significant reduction in dynamic loading (see Figure 3.5). Although the cause of the variations in performance by the same suspension is not possible to ascertain, it appears to be related to the maintenance and fitting of the spring. This may have implications for vehicle maintenance and pavement management.

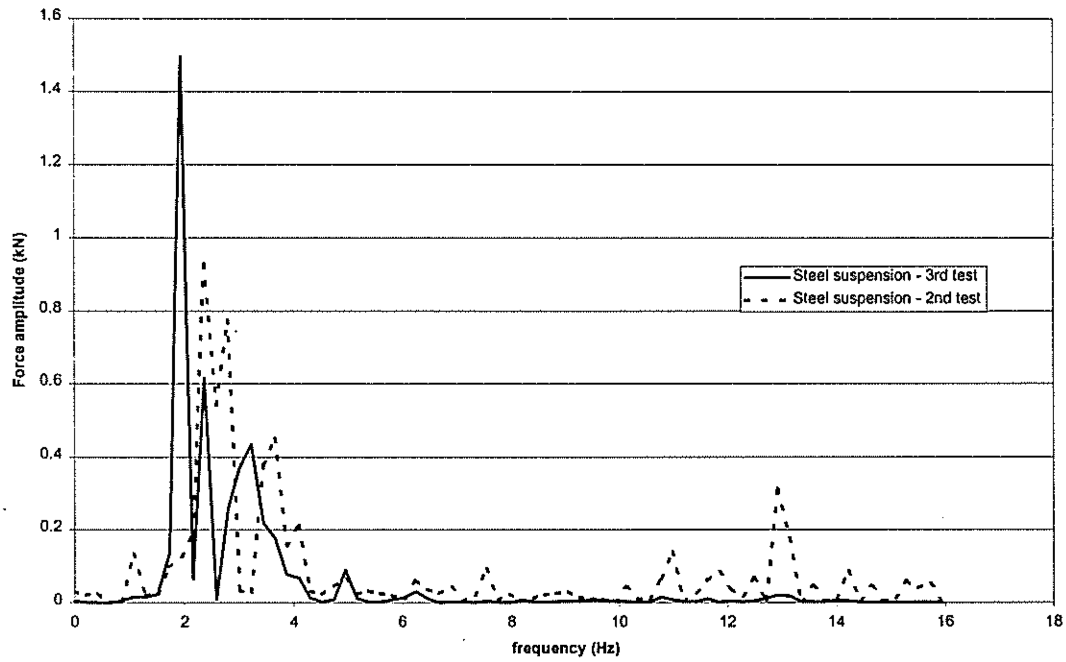

Figure 4.2 Comparison of steady speed wheel force spectra obtained for second and third tests.

At each measurement interval during the test,the applied dynamic wheel forces were measured for the SLAVE units travelling at test speed (45 km/h) in the wheel paths they were trafficking. At every second measurement interval the SLAVE units were moved laterally into the other wheel paths, and the dynamic loads were measured. From these dynamic loads the DLC values were calculated. As the dynamic loads represent the vehicle's response to the pavement surface profile, these wheel force measurements are a surrogate measure of pavement roughness.

Figure 4.3 shows the progression of the DLC of each vehicle-suspension configuration in each wheel path with increasing load cycles. At the start of the test, both wheel paths generated approximately equal dynamic load levels for a given suspension configuration. As the test proceeded, the level of dynamic loading in both wheel paths increased indicating an increase in roughness. However, the rate of increase was significantly less on the inner wheel path, which was trafficked by the vehicle with air suspension. The rate of increase in DLC of the air-suspended vehicle was also less than that for the steel suspension on both wheel paths, indicating that the air suspension was better able to accommodate increased pavement roughness.

This pattern of behaviour is very similar to that observed for the second pavement test (see Figure 3.5).

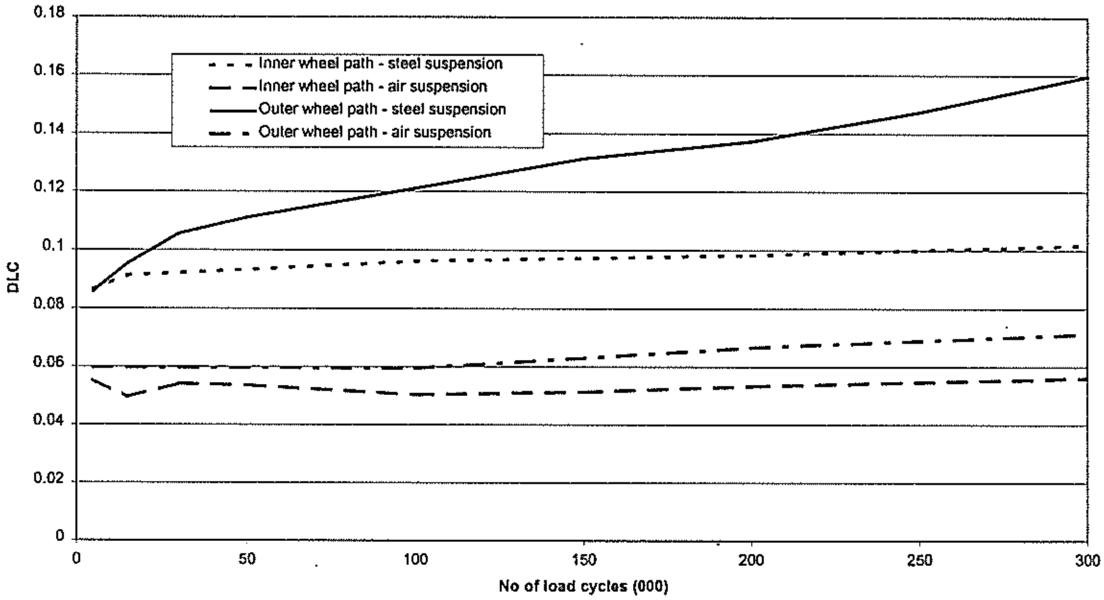

Figure 4.3 Change of dynamic wheel forces (DLC) with increasing number of load cycles for inner and outer wheel paths and both types of suspension.

4.4.4 Pavement Strains

During the pavement construction, 14 sets of Bison soil strain coils were installed in the pavement, 8 pairs in the outer wheel path and 6 pairs in the inner wheel path. The coil pairs were distributed between stations 7 and 10 inclusive. The coil pairs were installed in the bottom 100 mm of the basecourse layer, the top 100 mm of the subgrade, and at a depth of 100 to 200 mm below the surface of the subgrade. In the early stages of trafficking, a large number of coil pairs became unusable because of damage sustained to either the coils or the cabling. The coil pairs surviving at the completion of trafficking were located in the lower subgrade (100 to 200 mm below the surface of the subgrade), at stations 8 and 9 in the outer wheel path, and at station 10 in the lower subgrade in the inner wheel path. The strain values recorded were high (between 4000 and 6000 microstrain for the top of the subgrade), but these high values did not seem to affect the response or long-term performance of the pavement. One possible explanation for this is that the material used for the subgrade (a silty clay) is very elastic in its response to dynamic loading, but sustains very little plastic strain for each loading cycle. It is noteworthy that although the AUSTROADS design model resulted in a predicted life of 350,000 load cycles, which was very much in line with what was observed, the subgrade strain on which this life was based was very much lower than the measured values (1420 microstrain v 4,000 to 6,000 microstrain). The measured pavement strains are shown in Figures 4.4 and 4.5 for the inner and outer wheel paths respectively.

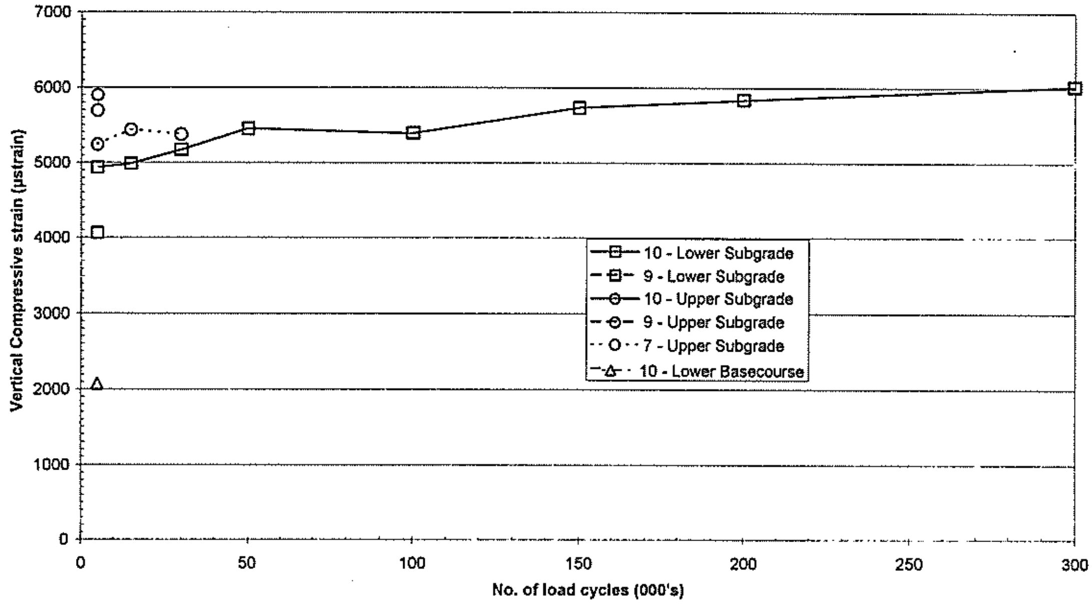

Figure 4.4 Pavement strains on inner wheel path at stations 7, 9 and 10.

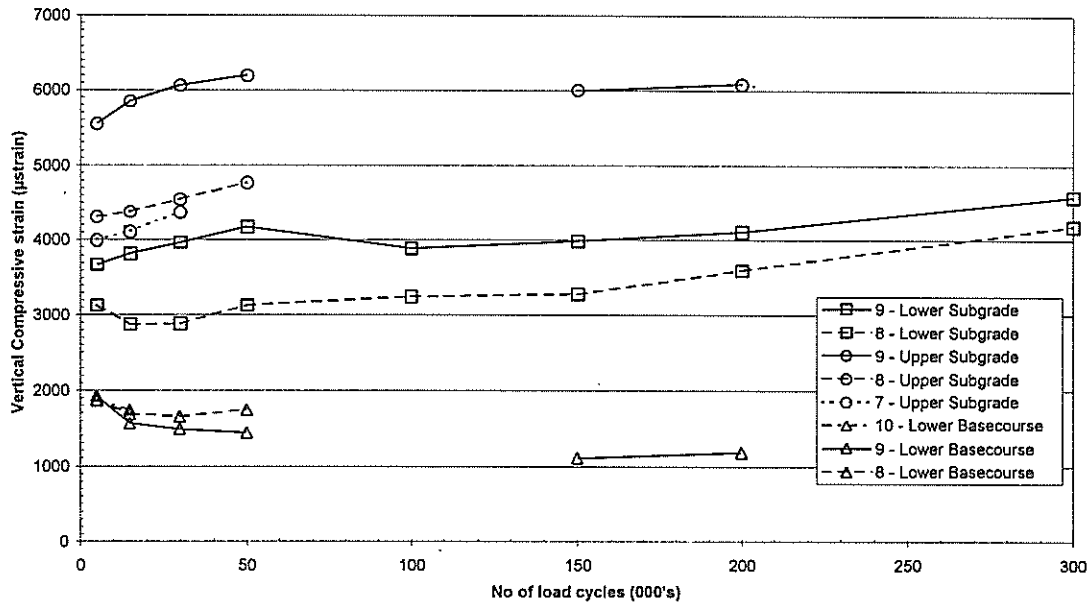

Figure 4.5 Pavement strains on outer wheel path at stations 7, 8, 9 and 10.

4.4.5 Surface Profile Changes and Rutting

The changes in dynamic loading discussed in Section 4.4.3 are generated by changes in the pavement surface profile. As in the previous two tests, there were two types of profile measurement: transverse profiles, which were measured at each station using the CAPTIF profilometer, and longitudinal profiles, which were measured using a laser profiler developed for CAPTIF by ARRB Transport Research during the previous test.

Figure 4.6 shows the changes in transverse profile for a typical station as the number of load cycles increased. Because of the number of traces plotted in this figure it is difficult to see each one clearly but the general trends are obvious. As with the previous test, the initial rate of change is much higher than the rate of change later in the test. The number of load cycles between consecutive traces is much lower early in the test, so this effect is even stronger than an initial assessment of the figure might indicate. The three vertical lines on the figure show the centrelines of the inner wheel path (left), the whole track (centre), and the outer wheel path (right line).

As explained previously the centreline of the track is not trafficked by either vehicle during the loading programme except for the initial conditioning laps. (It is also subjected to a few load cycles while the vehicles are transferred from one wheel path to the other for different measurements during each measurement phase.) Although most of the vertical surface deformation recorded on this centreline occurred during the conditioning laps (i.e. the first 5,000 load cycles), some further deformation does occur during loading. This can be attributed to the combined effect of the rut formation in both wheel paths.

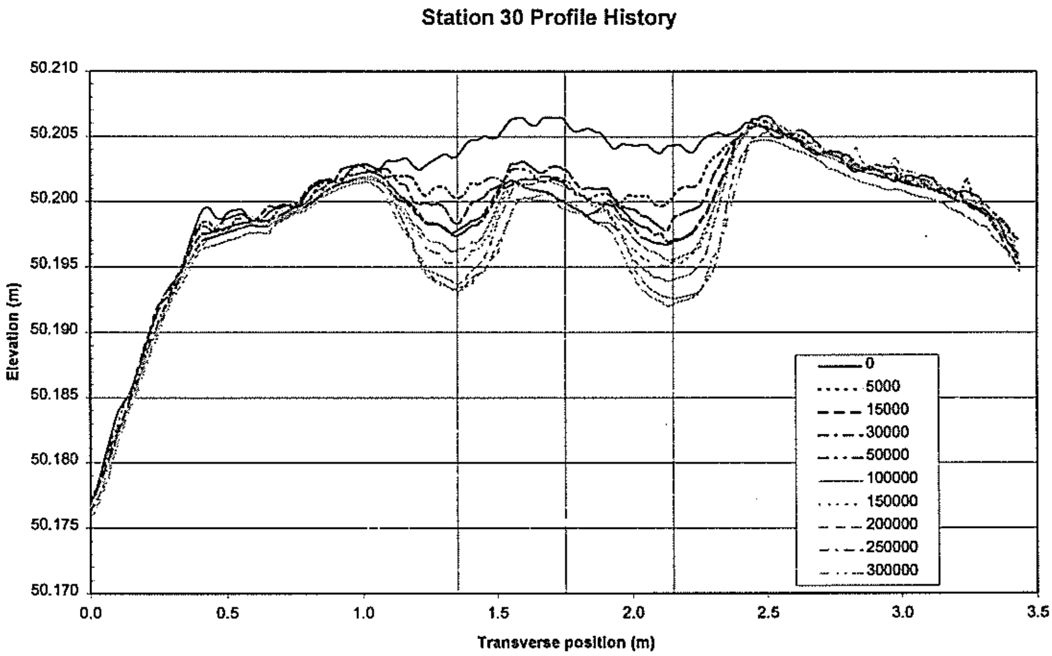

Figure 4.6 Progression of rutting at station 30 with increasing number of load cycles.

The transverse profile changes in each wheel path can be characterised using either vertical surface deformation or rut depth. Because the transverse profiles at CAPTIF are measured relative to absolute reference points around the side of the tank, it is very easy to determine vertical surface deformation by taking the difference between the current profile and the profile measured at the start of the test.

For in-service pavements the more usual approach is to measure rut depth, which is the vertical distance from a straight edge placed across the rut to the bottom of the rut. Determining the absolute elevation change is not necessary.

This method can also be applied numerically to the CAPTIF transverse profile data. However, because the elevation changes in the untrafficked zone between the two wheel paths is contributed to by the ruts in both, some thought is required in interpreting these results.

Figure 4.7 shows the changes in the mean, maximum value and standard deviation of the vertical surface deformation in each wheel path as the test progressed. After the initial rapid changes, steady increases in all three quantities were recorded as the test proceeded. Very little difference shows in the mean value of surface deformation between the two wheel paths. The standard deviation, however, does show a difference, which widened as the test proceeded.

The outer wheel path, which was trafficked by the SLAVE with steel suspension, has a higher value and hence greater variation in surface deformation around the track. This is consistent with the result for wheel force in Section 4.4.3, which also implied that the outer wheel path was getting rougher than the inner.

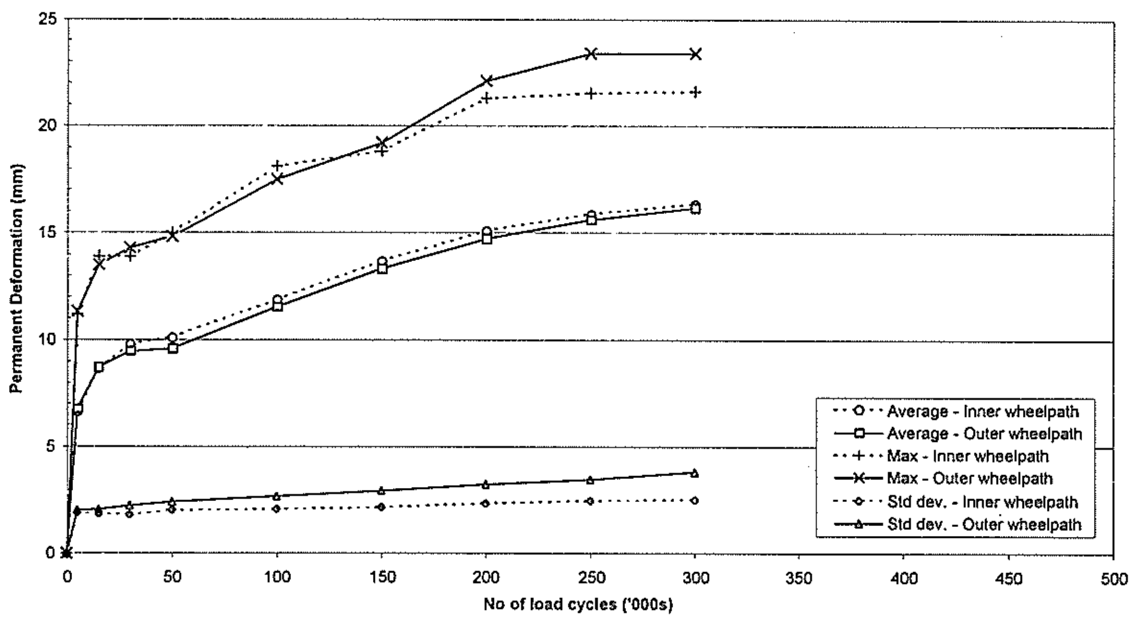

Figure 4.7 Progression of vertical surface deformation with increasing number of load cycles.

From normal statistical principles it would be expected that the maximum surface deformation would show the same trend as the standard deviation. This is generally supported by Figure 4.7 although the maximum deformation traces are not as well-behaved. The reason for this is that both the means and standard deviations are calculated from 58 measurement values while the maximum is based on only a single value. The means and standard deviations thus have a much smaller measurement error. The deviations of the maxima curves from a smooth trend are of the order of 1 mm or less for measurements on an asphalt surface containing stone chips of up to 7 mm.

Instead of surface deformation rutting can be plotted as shown in Figure 4.8. A major difference over surface deformation is that the rutting is not zero at zero load cycles. Although an initial interpretation might say that the rutting in the outer wheel path is greater than that in the inner, and therefore the higher dynamic loads generated by the steel suspension cause more rutting, a closer look shows that the difference is approximately constant as the loads increase. Thus the change in average rutting is about the same for both wheel paths.

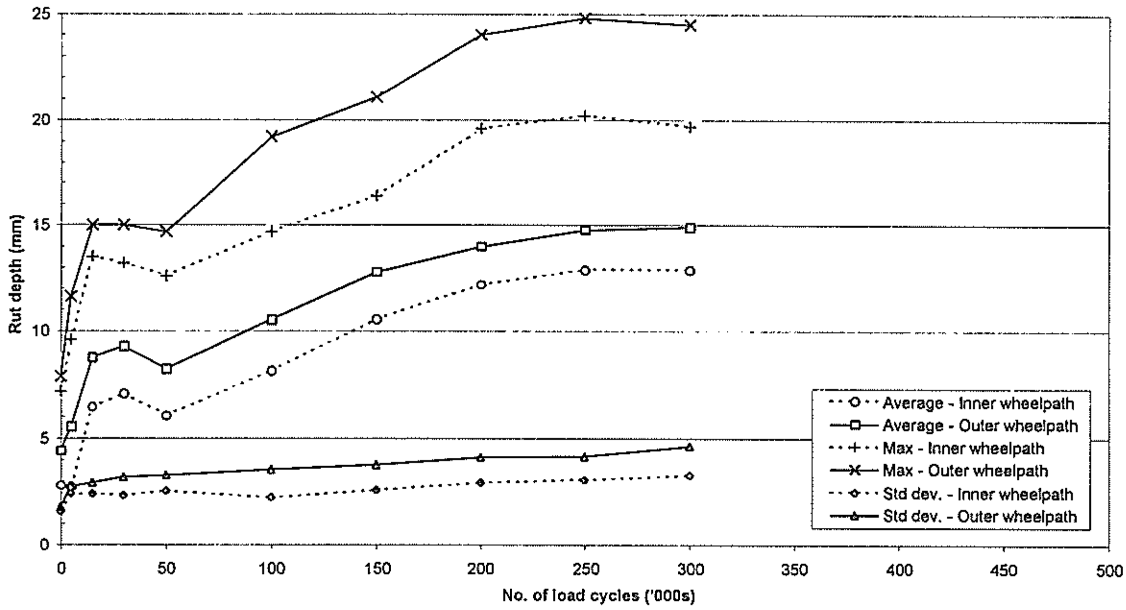

Figure 4.8 Progression of rutting with increasing number of load cycles.

The standard deviation does not depend on the mean level and so this measure shows the same result as the standard deviation of the surface deformation. The maximum rut depth of the outer wheel path is greater than that of the inner and this difference increases with load cycles. Although a part of this difference can be attributed to the difference in the mean values, the difference between the maxima is increasing as well and so the outer wheel path is becoming more variable with larger ruts.

Based on the pre-set failure criterion of a maximum rut depth of 25 mm, the outer wheel path is on the verge of achieving this while the inner wheel path would probably take at least another 100,000 load cycles or more to do so. However, as suggested in the previous paragraph, the difference in the means at the starting point should be taken into account. Furthermore there is some indication that the rate of increase in rutting in both wheel paths was slowing considerably when the test was terminated, and that both wheel paths may have continued for much longer still. Although it does appear clear that the outer wheel path will achieve the failure criterion first, it is not possible to accurately estimate the difference in life between the two wheel paths.

As with the previous tests, the cross-correlation functions between these surface deformations and the dynamic wheel forces were calculated. Figure 4.9 shows these functions for the two wheel paths. The results are similar to those from the previous tests, with a moderate correlation between surface deformations and wheel forces for the outer wheel path, which was subjected to the higher dynamic loads, and a weak correlation for the inner wheel path, which had low dynamic loads. The periodicity in the cross-correlation function for the outer wheel path has a wavelength of approximately 6.5 m, which at the test speed (12.5 m/s) corresponds to just under 2 Hz. This matches the EC bump test response of this suspension (see Figure 4.1). The inner wheel path does not show this clear periodic behaviour.

The other probable influence on the surface deformation is the structural capacity of the pavement. Calculating the correlation coefficient between the peak deflection at each station, from the CAPTIF deflectometer measurements after construction, with the surface deformation gives a value of 0.64 for both the inner and outer wheel paths. That is, there is some correlation between the structural capacity of the pavement and the surface deformations, and it is about the same for both wheel paths.

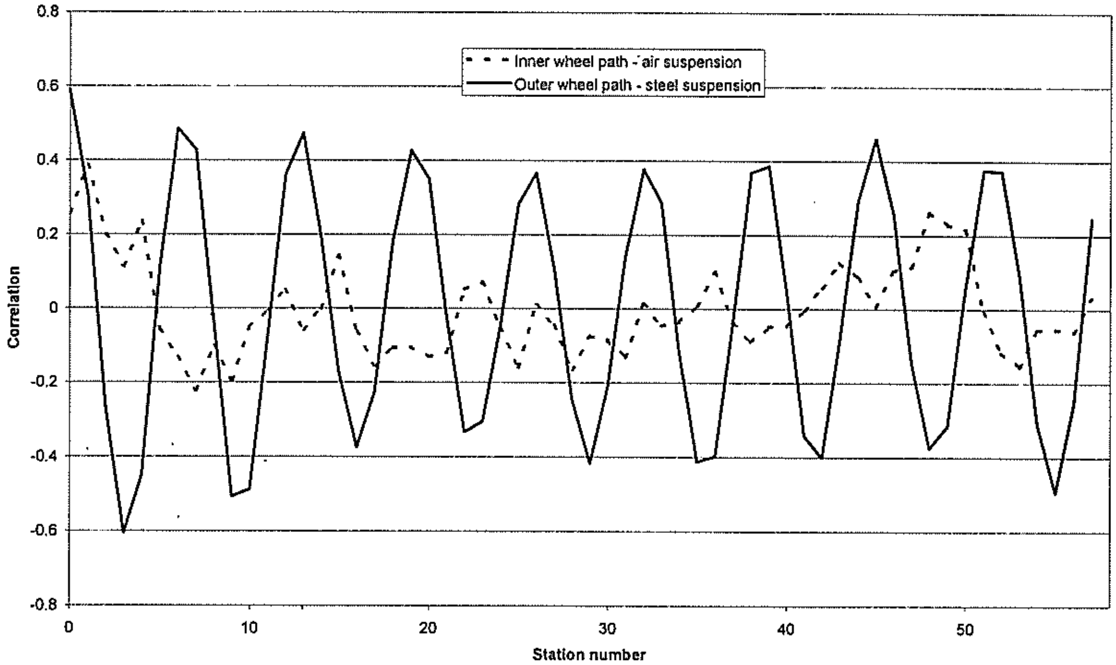

Figure 4.9 Cross-correlation between permanent deformation and dynamic wheel force for inner and outer wheel paths and both types of suspension.

Longitudinal profiles were measured in each wheel path at each measurement interval using a laser-based profiler, which is mounted on one of the SLAVE units. At approximately every second measurement interval, profiles were also measured along the track centreline (midway between the two wheel paths), at 0.4 m inside the inner wheel path, and at 0.4 m outside the outer wheel path. These profiles provide elevation data at a longitudinal spacing of approximately 0.05 m which is much finer spacing than provided by the transverse profiles. The absolute elevation of the longitudinal profile is determined by referencing the start point to the corresponding point on the transverse profile at station zero. The longitudinal profiles provide an alternative data source for calculating the pavement permanent deformations.

A typical use of longitudinal profile data in pavement management is to use them to calculate a measure of roughness such as the IRI. Figure 4.10 shows the changes in IRI for the two wheel paths as the test progressed. Note that the data for the inner wheel path at 300,000 loads are missing. As discussed in the results of the previous two tests (Sections 2.4 and 3.4), the use of IRI in the CAPTIF environment has some limitations. The radial screeding used in pavement construction results in the inner wheel path having a higher IRI than the outer, although this is offset by the vehicle speed variation with radius so that similar dynamic wheel forces are produced in both wheel paths. This again is the case for this third test. As with the earlier tests initial IRI is relatively high. The initial loading cycles result in some smoothing of the pavement and a reduction in IRI, followed by a slow but steady increase.

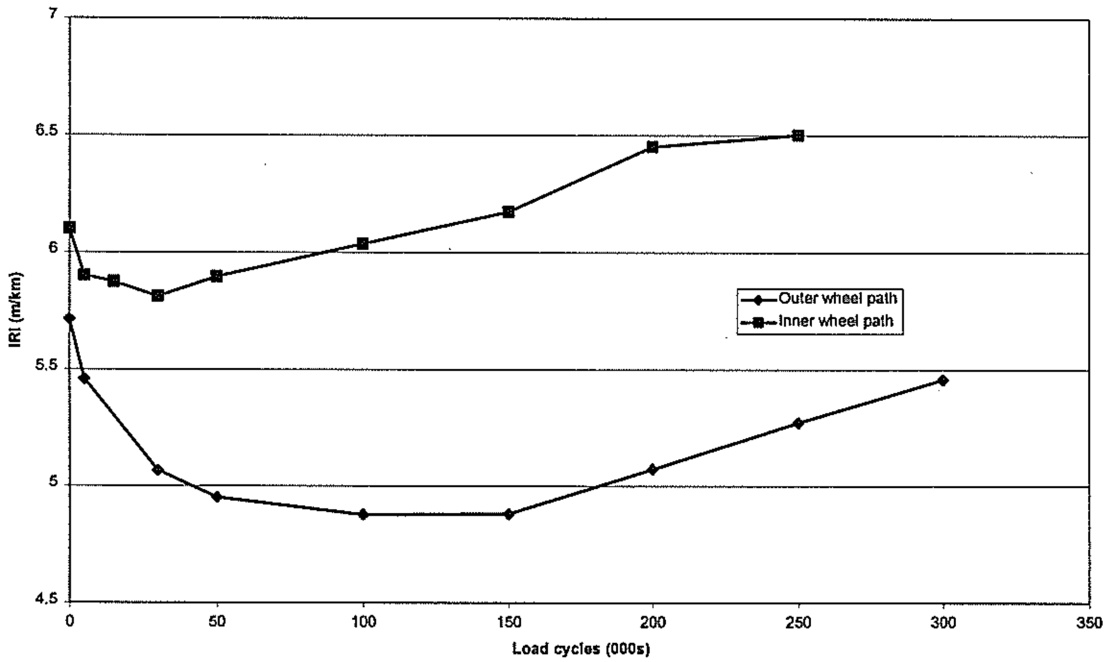

Figure 4.10 Changes in IRI (m/km) with increasing number of load cycles.

Although the wheel force and transverse profile data presented above (Figures 4.3, 4.7, 4.8) indicate that the outer wheel path is increasing in roughness more than the inner wheel path, this is not clearly indicated by the IRI. From the IRI the initial smoothing appears to continue for longer on the outer wheel path, although the deterioration in roughness from 150,000 load cycles onwards does appear to be faster in the outer wheel path.

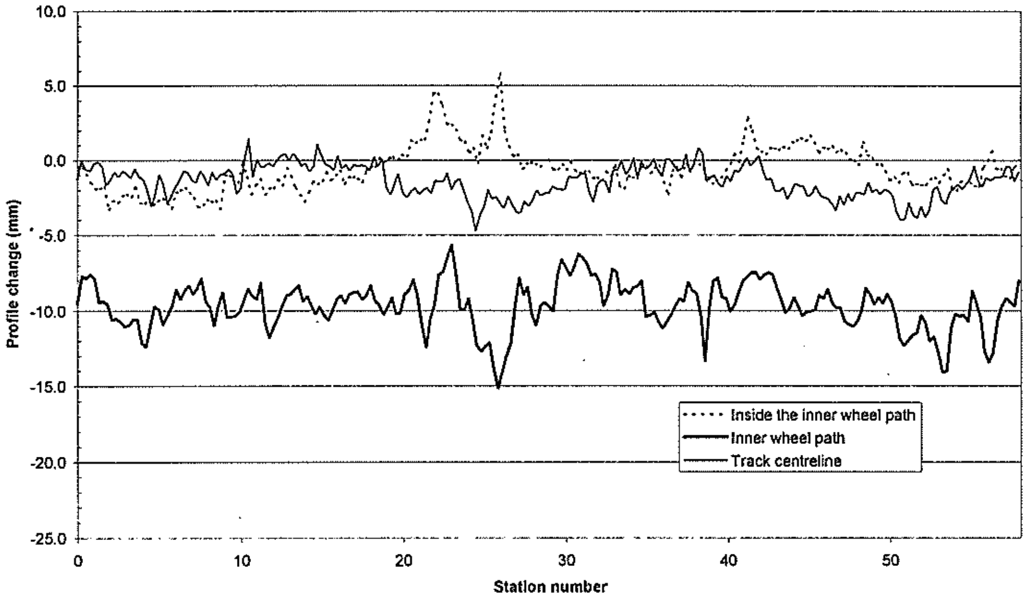

Figure 4.11 Profile changes for inner wheel path between 5,000 and 250,000 loads.

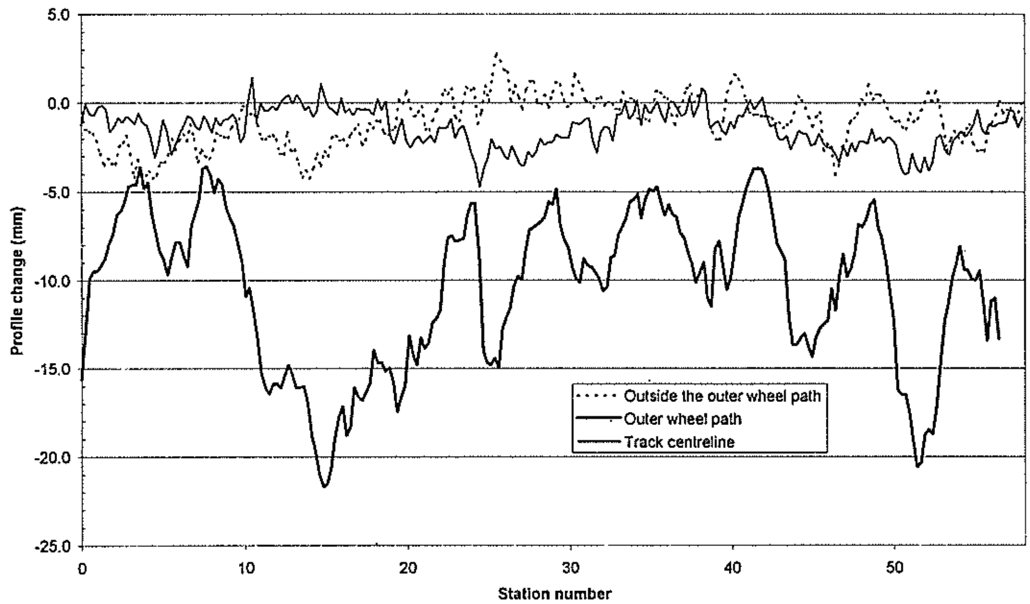

Figure 4.12 Profile changes for outer wheel path between 5,000 and 250,000 loads.

By taking the difference between the profiles at the end and start of the test the permanent surface deformation can be calculated. Figures 4.11 and 4.12 show the profile changes between 5,000 load cycles and 250,000 loads for the inner and outer wheel paths respectively. As can be seen, the untrafficked tracks between and outside the wheel paths show very little change. The two wheel paths themselves show a similar mean change (about 10 mm) but the variation in the outer wheel path is much greater than that for the inner. These results are consistent with the permanent deformation results calculated from the transverse profile data, although the difference in variability is much clearer in this case.

As with transverse profile-based deformations, the cross-correlation functions between dynamic wheel forces and profile changes can be calculated as shown in Figure 4.13. This figure can be compared with Figure 4.9, which shows the cross-correlation between surface deformation and dynamic wheel force. An obvious difference between these two figures is that the correlation is of opposite sign. This is because surface deformations are positive while the profile changes are negative. Otherwise the figures are very similar.

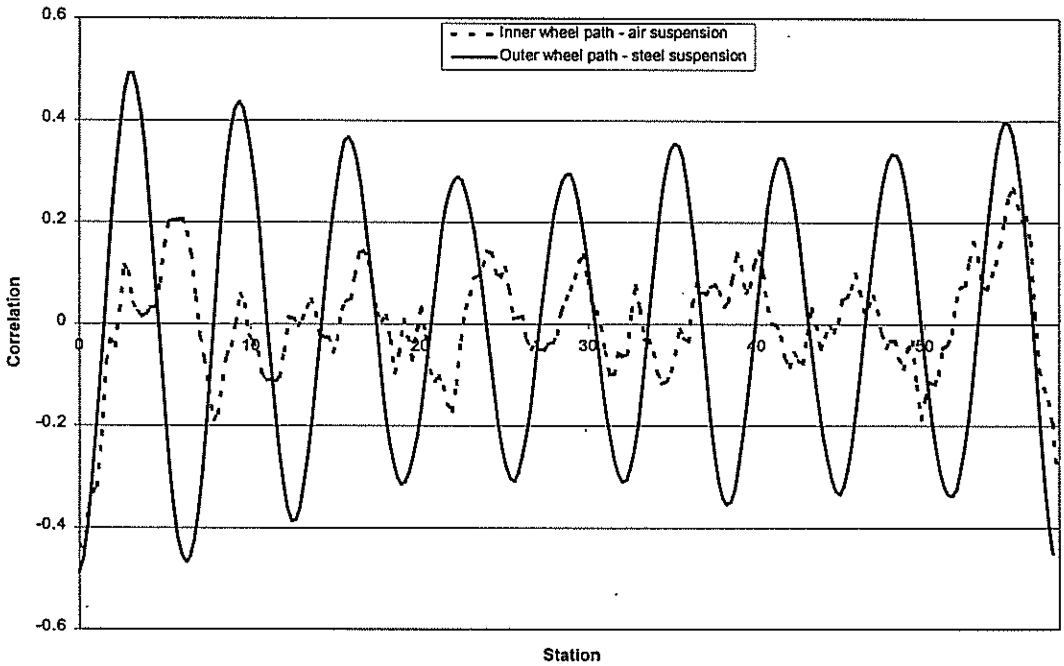

Figure 4.13 Cross-correlation between profile changes and dynamic wheel forces, for inner and outer wheel paths and both types of suspension.

The correlation value for the inner wheel path at zero offset is similar to that for the outer but it does not have the same strong periodicity. This is a reflection of the better damping of the air suspension. With the steel suspension the cross-correlation has strong peaks, not only at zero offset but also at offsets corresponding to an integral number of wavelengths (the wavelength corresponds to the vehicle speed divided by the natural frequency of the suspension). If this suspension is excited it will continue to bounce and apply a sequence of force peaks at one wavelength spacing. The better damping on the air suspension keeps the effect more localised.

4.4.6 Post-mortem Excavations and Analysis

At the completion of loading, four trenches were excavated across the entire width of the track to determine the change in material properties after loading. The trench locations were based on the minimum, average and maximum rut depths in both the inner and outer wheel paths. Table 4.4 outlines the selection criteria for the trench locations.

Table 4.4 Selection criteria for post-mortem trench locations.

Station	Wheel Path	Selection Criteria
4	Inner	Maximum rut depth
4	Outer	Average rut depth
31	Inner	Minimum rut depth
41	Outer	Minimum rut depth
52	Inner	Average rut depth
52	Outer	Maximum rut depth

In previous projects, the cutting of the surfacing for the trenches has been carried out with a diamond-tipped, water-cooled concrete saw. Because of time and budget constraints, it was decided to use an electric breaker fitted with a wide chisel bit to cut through the thin surfacing layer. This method of cutting the asphaltic concrete made it difficult to accurately determine the location of the interface between basecourse and asphaltic concrete layers. Thus the values for the asphaltic concrete deformation are really only estimates. However, data from previous projects where the asphaltic concrete surfacing had been cut by a saw were analysed. From this it was determined that, on average, the thickness of the surfacing layer decreased by 5% over the duration of the project. This indicates that the estimates are reasonable. The results of the trenching (Table 4.5) show that the deformation occurring in the subgrade range from 30.5 to 51.9% of the total surface deformation.

Table 4.5 Contribution of layers to pavement deformation (with estimated AC deformation).

Wheel Path	Station							
	4		31		41		52	
Inner	(mm)	(%)	(mm)	(%)	(mm)	(%)	(mm)	(%)
AC	1.24	4.8%	1.28	8.1%	1.48	8.9%	1.30	6.9%
Basecourse	15.51	60.2%	6.98	44.3%	9.53	57.7%	8.20	43.7%
Subgrade	9	35.0%	7.50	47.6%	5.50	33.3%	9.25	49.3%
Total	25.75		15.75		16.50		18.75	
Outer								
AC	1.43	5.9%	1.44	7.5%	1.45	10.0%	1.46	5.6%
Basecourse	12.08	50.3%	7.81	40.6%	6.30	43.4%	16.79	64.0%
Subgrade	10.5	43.8%	10.00	51.9%	6.75	46.6%	8.00	30.5%
Total	24		19.25		14.50		26.25	

Particle size distribution tests, carried out on samples of the basecourse removed from the trafficked regions in the trenches, showed no significant change in the grading of the basecourse. Results from Loadman tests performed on the exposed basecourse surfaces showed that the elastic modulus values increased by 20-30 MPa in most cases. Getting consistent results with the Loadman was difficult because of the uneven surface of the exposed basecourse. The results from the Loadman tests on the subgrade surfaces showed no significant change from the as-constructed values.

4.5 Conclusions

- As with the second pavement, this pavement substantially exceeded its design life. This phenomenon has been observed with other pavement tests at CAPTIF. The dynamic loading regime applied during these tests is expected to be more damaging than that applied by the New Zealand heavy vehicle fleet because of the high level of repeatability of the loading pattern. The main differences between the CAPTIF environment and the in-service situation are that the CAPTIF pavement is protected from precipitation and direct sunlight, and that the tank in which the pavements are constructed prevents the ingress of water into the structure.

- A further factor is that the pavements were constructed with a high level of quality assurance testing and were considerably more uniform than in-service structures. However, the surface roughness of the pavements as-constructed was higher than would be acceptable for a newly constructed highway, although to some extent this is a reflection of the appropriateness of using IRI in the CAPTIF environment.

- The dynamic wheel forces generated by the two suspension systems used in this third test were substantially different, in that the multi-leaf steel suspension generated dynamic loads that were about twice as high as those generated by the air suspension. This difference was not as great as that observed in the second pavement test, even though it incorporated the same suspension configurations with the same static load. The reason appears to be an improved performance by the steel suspension, which may be the result of the maintenance and servicing undertaken on CAPTIF between the two tests.

- The trends in dynamic wheel forces as indicated by DLC provide a good indicator of the progression of pavement wear at CAPTIF and are better than IRI for this purpose. The main drawback is that DLC does not provide any absolute reference points for comparison with other pavements.

- The critical mode of wear for this pavement is permanent deformation at the pavement surface. This can be determined from the transverse or longitudinal profile measurements. Other related measures such as rutting can also be calculated.

- The mean level of surface deformation was approximately the same for both wheel paths. However, the variation in the surface deformation is much greater for the outer wheel path, which was subjected to the higher dynamic loads. Correlation analysis showed that the profile changes in both wheel paths were moderately correlated with both the dynamic wheel forces and the initial structural capacity of the pavement. The two factors together determine the pattern of surface deformation that occurs.

- The implication is that constructing the pavement to a high degree of uniformity and using suspensions that result in low dynamic loads will generate a uniform distribution of surface deformation. A uniform surface deformation means a slower increase in roughness and a more constant level of rutting, both of which effectively increase the life of the pavement.

5. ANALYSIS OF THE TESTS

5.1 Introduction

In Sections 2-4, results from each of the three tests have been presented in isolation from each other. The aim of this chapter is to bring these findings together as a coherent overall picture. Some of the findings are common across all three pavement tests and thus can be used to support more general conclusions, while others differ between tests. These inconsistencies are identified and hypotheses are proposed to explain them.

5.2 Dynamic Wheel Forces

For each test the intention was that the SLAVE units would have identical static loads and significantly different dynamic loads. The first test was conducted immediately after completing the modification to CAPTIF to accommodate the parabolic leaf spring suspension. As this was a more modern suspension design than the multi-leaf steel, it was expected to be more road-friendly. Parabolic leaf springs have substantially less interleaf contact and do not use this for damping, but instead require a viscous damper to be fitted. As initially configured, however, this suspension did not generate dynamic loads that were consistently lower than the multi-leaf spring for different excitation levels. After considerable time was spent investigating this apparent lack of performance without success, it was decided to proceed with the test because of time pressure to have the facility available for the OECD DIVINE test, which was then being planned. Although it was not possible to rank the two suspensions, their behaviours were quite different and thus a difference in the performance of the pavement under the two types was anticipated. Subsequent to the test, the reasons for the poor performance of this suspension were determined and in retrospect were quite simple. The spring was designed for a 60 kN load and therefore was too stiff for the 37 kN wheel load being used. As well the damping of this suspension was too low. This was partly because high spring stiffness leads to a higher value for critical damping. Hence for a given damper the actual damping is a lower proportion of the critical damping value..

Both these factors are important to the in-service situation in New Zealand. Heavy vehicle prime movers are all sourced overseas, often from countries with higher axle load limits than New Zealand. Suspensions designed for these higher axle loads do not necessarily perform well in terms of road-friendliness or ride. The importance of adequate damping for road-friendliness has been identified previously by a number of authors (de Pont 1996, Hahn 1985, OECD 1992).

For the second and third tests the SLAVE units were fitted with the same static wheel loads (49 kN) and the same pair of suspension configurations: the multi-leaf steel and an air spring with viscous damper. The performance of the air-spring suspension was consistent across both tests. It generated relatively low dynamic loads, which increased only slightly as the pavement roughness increased. The multi-leaf steel spring produced significantly higher levels of dynamic loading which increased markedly with increasing pavement roughness. Perhaps more surprisingly it generated significantly higher levels of dynamic loading for the second pavement test than for the third, even though in the second test the pavement roughness for this pavement as indicated by the IRI was lower.

A clue to the reason for this occurs in the second pavement test when, just before 1.5 million load cycles, the axle assembly on this suspension was disassembled for a wheel bearing repair. On re-assembly there was a dramatic reduction in dynamic loads. Between the second and third pavement tests the CAPTIF was extensively overhauled. It is likely therefore that this maintenance programme, where the spring pack will have been disassembled, cleaned and re-assembled, resulted in the improved performance. It is noteworthy, however, that the EC bump test, which is used at CAPTIF to characterise and monitor suspension performance, showed no significant changes. The implication for in-service operations is that there can be quite a difference in the dynamic loads produced by a multi-leaf steel suspension depending on servicing and maintenance. In practice very little servicing is done to these spring packs.

Correlation analysis showed a moderate correlation between surface profile changes and dynamic wheel force. In general, this correlation was stronger with higher dynamic wheel loads. This implies that a more road-friendly suspension producing lower levels of dynamic loading results in a more uniform surface deformation. A completely uniform surface deformation would result in no change in roughness, so this does reduce the need for maintenance. Equally, a more even pattern of rutting means that the time before any part of the rut exceeds some critical value will be longer.

During these tests each wheel path was subjected to loading by only one vehicle, and hence subjected to a very specific pattern of dynamic loading. On in-service highways the loads are applied by a fleet of vehicles, and hence there is some distribution of the pattern of loading. However, Research Element 5 of the OECD DIVINE project showed that the vehicle fleet as a whole applies a spatial pattern of loading. It might reasonably be expected (though it has not been proven) that a fleet of vehicles with road-friendly suspensions would generate a pattern of loading with smaller variations, than a fleet of vehicles with less road-friendly suspensions.

5.3 Structural Variability

The pavements constructed at CAPTIF for these tests were intensively monitored and tested during construction. It was critical to the experiments that the difference in the pavement structure between the inner and outer wheel paths was minimal. These two factors resulted in pavements that were uniform in terms of their structural capacity.

In spite of this care there were some difficulties. The first pavement failed prematurely because of inadequate compaction of the basecourse layer. This occurred because of difficulties in operating a vibratory roller that had to be constantly turning because of the circular track. This problem has since been resolved by the use of a flat plate compactor. This problem did not affect the uniformity of the structure in that the whole pavement was inadequately compacted.

A more surprising problem occurred with the second pavement when a severe localised rut appeared in the outer wheel path very early in the test. The material density and structural capacity measurements taken during construction and immediately afterwards showed absolutely no evidence of any weakness or fault in the pavement at this location. Structural capacity measurements (FWD) made immediately before the formation of the rut did show a weakness. After forming, the rut stabilised and the pavement in this area performed similarly to the rest. During the post-mortem excavations special attention was given to this area to determine the cause of the problem. The rut was caused primarily by compaction of the basecourse layer, but there was no evidence of any material weakness or moisture content changes or any other factor to explain what had happened. The only known factor was that the rut was immediately next to one of the main doors to the facility and that, in the past, failures have sometimes occurred near these main doors.

Across all three pavements there was a moderate correlation between the pavement wear and the initial structural variability. This indicated that minimising structural variability will keep the pavement wear pattern more uniform and reduce the maintenance requirements.

5.4 Pavement Performance

New Zealand-style thin-surfaced pavement structures are based on a design model that assumes that the surface layer provides water-proofing and a running surface, but does not contribute to the pavement strength. The unbound basecourse layer is the main structural element, and its role is to distribute the tyre loads so that the underlying subgrade is not over-stressed. For this design model, the main pavement wear mechanism is that the subgrade undergoes permanent deformation, which manifests itself at the surface as rutting. Rutting, vertical surface deformation, longitudinal profile changes and even roughness changes are all measures that can be used to indicate the extent of this wear.

With thicker AC pavements like the pavement used for the second test, the AC surface layer is an important structural component. In addition to the subgrade deformation of the thin-surfaced pavements, these pavements have two other main wear mechanisms.

One of these is rutting caused by visco-elastic flow of the asphalt. This adds to the rutting caused by subgrade deformation, and so the various measures mentioned above reflect the combined effect of these two forms of wear.

The second wear mode is cracking of the asphalt layer caused by fatigue damage. These cracks would be expected to initiate at the bottom surface of the asphalt and propagate to the surface.

These are the theoretical wear mechanisms. In practice the effects of the wear such as surface deformation, rutting, road roughness, surface cracking, structural capacity, are measured during the pavement's life and the actual mechanisms are not able to be determined until the post-mortem stage when the pavement can be excavated.

The first pavement underwent substantial permanent deformation of the surface. There was a substantial difference in the mean level of profile change between the two wheel paths between 10,000 load cycles and 35,000 load cycles, with the value for the inner wheel path being 67% higher than the outer. However, the profile change between zero loads and 35,000 loads has a similar mean for the two wheel paths. The variability in profile change as given by the standard deviation was higher for the inner wheel path in both cases.

This pavement was the only one of the three that failed and it failed prematurely. The surface deformation that occurred was caused by compression in the basecourse layer accompanied by lateral movement of material in this layer. This is a failure mechanism that should not occur and was attributed to inadequate compaction of the basecourse layer during construction. The test was valuable in demonstrating the relationship between dynamic loads and surface profile and profile change. It also reinforced the need for quality control at construction.

The performance of the second pavement was complicated by the formation of a localised rut early on in the test in the outer wheel path. In spite of considerable investigative effort, this rut was never satisfactorily explained. Much of the data analysis was done for two data sets, one including all data and the other excluding data from this rutted section. If the rut is excluded then the mean level of wear in both rutting and cracking was very similar in both wheel paths. However, the variability of the wear as indicated by the standard deviation was significantly higher in the outer wheel path, which was subjected to higher levels of dynamic loading. Similar findings were obtained for the surface deformations on the third pavement.

The post-mortem analyses showed some surprising outcomes. Although all three layers experienced some deformation most of the deformation occurred in the unbound basecourse layer. There are some differences in detail between the deformations in these last two pavements, but this observation applies to both. In the second pavement where cracking wear was also an issue the cracks for the most part were initiated at the top surface of the asphalt. This is a departure from the theoretical model of pavement behaviour but has been observed elsewhere.

The patterns of pavement wear were in all cases moderately correlated to both dynamic wheel forces and to the pavement structural capacity. To some extent the correlation with wheel forces was stronger when the dynamic wheel forces were higher.

Conversely in the second test particularly, the correlation with structural capacity was higher when the dynamic wheel forces were lower. For the third pavement these correlation magnitudes were similar for both wheel paths.

5.5 Conclusions from Analysis

The mean level of pavement wear seems to be independent of dynamic wheel load. At first glance this would seem to indicate that a first power rather than a fourth power relationship applies between wheel loads and pavement wear. The Eisenmann road stress factor shown in Section 1.3 gives the expected value (or mean) pavement wear for a fourth power relationship. Based on the levels of dynamic load that were observed, we find the expected difference in the mean wear for the third pavement test is only between 2% and 13% depending on whether the dynamic loads at the start or finish of the test are used.

The differences for the second pavement test are somewhat higher particularly near the end of the test when the differences in dynamic load were greater. If a lower power is used these differences are much smaller still. From the data obtained it is not possible to say with confidence that there is not a difference of say 10% in the mean wear between the inner and outer wheel paths. So we cannot say with confidence that the relationship between load and wear is not fourth power. It appears unlikely that a power greater than four is correct, and quite possibly that a power less than four may be correct.

The pattern of pavement wear is related to both the dynamic loading and the structural variability of the pavement. Reducing both these factors should lead to a more uniform pattern of pavement wear and hence a reduction in the need for maintenance to repair the worst affected areas.

The results also confirm that pavement response to loading is greater than that predicted by response models based on linear elastic theory. They also confirm that the subgrade strain criterion alone is inadequate for predicting the performance and life of a flexible pavement under cumulative wheel loading. The vertical compressive strain in a thin-surfaced unbound granular pavement can be equal in magnitude to the vertical compressive strain in the subgrade, and thus can be a significant contributor to fatigue of the surfacing layer and permanent deformation. Yet the strains in the basecourse are not explicitly considered to be critical in the AUSTROADS pavement design procedures.

In practice and in the projects described in this report, shear and deformation within the unbound granular pavement layers are often the principal cause of failure. Failure through classical rutting caused by subgrade deformation is relatively rare. Beside the AUSTROADS Pavement Design Guide, no other road authorities in the world use a mechanistic design procedure for unbound granular pavements. The stress-strain characteristics of both the subgrade and unbound granular pavement layers should be an integral component of mechanistic pavement design.

6. CONCLUSIONS & RECOMMENDATIONS

Design life of pavements

Ignoring the first test pavement, which failed because of construction defects, it appears that pavements at CAPTIF last much longer (in terms of load cycles) than the design life predicted by many of the models, possibly up to three times or more as long. This phenomenon has been observed on previous pavement tests at CAPTIF. However, the AUSTROADS design guide, which was adopted in New Zealand in 1995, appears to predict design life values at CAPTIF of the right order of magnitude. The dynamic loading regime at CAPTIF is likely to be more severe than the in-service situation because the loads come from a single vehicle with a highly repeatable pattern of loads. Pavement construction at CAPTIF is highly monitored and is probably of a better standard of uniformity and consistency than in-service pavements, although data from in-service pavements in New Zealand are limited. The main differences between CAPTIF pavements and in-service pavements appear to be reduced construction variability, reduced environmental influences, and the accelerated testing which reduces aging effects. If these are the reasons for the increased pavement life then it implies that some pavement wear can be attributed to environmental and aging effects rather than loading. This has major implications for road pricing and user charges policies. A complication in this analysis is that the loading and environmental factors may interact with each other and may not be able to be separated in this way.

- **Recommendations**

Improved quality control and higher standards of uniformity and consistency during pavement construction should be promoted, as these will improve pavement performance. Some further investigation to quantify the benefits and costs may be needed.

The relative magnitude of pavement wear costs associated with traffic loading compared to those related to environmental factors should be determined. This is an important factor in allocating costs to users.

Pavement wear

The mean level of pavement wear observed for the last two tests appears to be independent of dynamic loading. While this might be interpreted as indicating a first power relationship between load and wear rather than a fourth power, this is not necessarily the case. Because the static load is the same for both SLAVEs in each test, the difference in mean wear that would be expected by applying a fourth power to the different dynamic loads is relatively small. Given the accuracy of the measurements, the possibility of a fourth power relationship cannot be conclusively eliminated. Lower powers (between one and four) would also fit the data. The AUSTROADS design guide, which was the best predictor of pavement life, is based on a load cycles versus subgrade strain relationship that has a power of 7.14.

As the relationship between load and strain used in AUSTROADS is linear, this implies a 7^{th} power relationship between load and wear. The observed mean levels of wear do not support a power as high as this.

- **Recommendation**

The relationship between vehicle loads and wear for thin-surfaced pavements needs further investigation. Although the results of this research do not eliminate the fourth power relationship, which is currently widely used, they do not provide strong support for it either.

Variations in pavement wear

The variations in pavement wear around the track are related to the dynamic loading and to the variations in structural capacity of the pavement. These relationships while expected have never been measured anywhere before. Lower dynamic loads result in a more uniform distribution of pavement wear. For the case where the type of pavement wear is vertical surface deformation, this reduces the increase in roughness. The associated rutting is also more uniform. The main factor driving pavement maintenance is the need to keep the surface roughness within acceptable limits.

Failure of the pavement structure is rare. Therefore lowering dynamic loads and improving the uniformity of pavement structures by better construction practices and quality assurance, will reduce the level of maintenance required to maintain satisfactory roughness levels. Quantifying this improvement is difficult. As explained in Section 3.4.4, IRI has some limitation in the CAPTIF environment, and the differences in the rate of change in IRI between the two wheel paths do not provide a clear measure of the reduced rate of roughness increase. Furthermore the CAPTIF situation is artificial in that each wheel path is trafficked by a single vehicle, while in-service pavements are trafficked by a fleet of vehicles. It is not clear whether a fleet of identical vehicles with road-friendly suspensions will produce a lower level of roughness increase, than a more disparate fleet with less road-friendly suspensions.

- **Recommendations**

Road-friendly suspensions generating lower dynamic wheel loads should be encouraged as their use will reduce pavement maintenance requirements.

Further research work is needed to quantify this effect and in particular the vehicle-fleet effects. Work in this area is in progress through the PGSF vehicle-road interaction programme.

Mechanisms of pavement wear and damage

The mechanisms of pavement wear and damage observed were different to those predicted in the design models. In particular, compression of the basecourse layer was a significant contributor to the permanent deformation.

The subgrade for these pavements was relatively strong (CBR 12) and this strength may be an influence. With the second pavement, most of the cracking in the AC layer initiated from the top of the layer rather than the bottom as predicted by most models.

- **Recommendations**

The mechanisms by which vertical surface deformation occurs on in-service pavements, particularly thin-surfaced structures, need to be resolved. This is fundamental to the design and management of the New Zealand roading network.

Crack initiation at the top of the AC layer has been reported elsewhere in the world and needs further investigation. However in the context of New Zealand's predominantly thin-surfaced pavements this is not a high priority.

Pavement design models

While the AUSTROADS design guide method generated pavement life predictions that matched the observed life of the CAPTIF pavements better than the other design models, it still has some weaknesses.

The pavement life was determined from calculated compressive subgrade strains using a linear elastic model. However, the calculated strains were much lower than the measured values. This suggests that the stress-strain relationships in the basecourse and subgrade are not the simple linear ones currently assumed.

- **Recommendation**

There is a need to investigate the reasons for the discrepancy between the measured and calculated strain values in the pavement. This stress-strain relationship forms the basis of the pavement design model.

On-going research

Objective one of the PGSF vehicle-road interaction programme is developing a Whole Life Pavement Performance Model (WLPPM) using data from these tests and from the OECD DIVINE for validation. This model includes the development of sub-models for fleet dynamic loading effects, pavement response to loading, and pavement damage. These three sub-models address the main research needs identified by this project. The validation and implementation of this model will be enhanced through good quality pavement performance data from in-service pavements.

- **Recommendation**

A long-term pavement performance monitoring programme should be established in New Zealand to provide good quality information on the performance of the roading network.

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