Strain Measurements In Flexible Pavements Under Heavy Vehicle Axle Loads

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ABSTRACT

Highway traffic in New Zealand is carried by flexible granular pavements surfaced with sprayed seals or thin layers of bituminous mixes. The primary design criteria for such pavements is to limit the vertical compressive strain in the subgrade, because excessive accumulations of the strain ultimately result in surface rutting and distress. The research described in this paper involves roads in service and New Zealand's facility for accelerated testing of the interaction between full-scale pavements and vehicles. Three recent projects at the test facility and an instrumented insitu road are described.

The primary finding in both the field and test track projects was that the actual strains in the subgrade were up to over three times that predicted by the pavement performance model, but the pavements still performed satisfactorily.

INTRODUCTION

New Zealand has a population of 3.3 million and an area of 268,675 km^2. The country's road network totals 100,000 km in length, of which 55,000 km have all-weather surfaces. At present, the maximum gross vehicle weight permitted on national highways is limited to 44 tonnes, and the maximum loads permitted for dual-tired single, tandem, and triple axle groups are 8.2, 15.0 and 18.0 tonnes, respectively. The New Zealand term for equivalent single axle load is Equivalent Design Axle (EDA). One EDA is defined as the unit of pavement wear caused by one passage of a 80 kN axle load on dual-tired wheels inflated to 550 kPa. Actual axle loads are related to the reference loads by the *"fourth-power rule"* (the exponent is 4.0). The national highway authority is called Transit New Zealand (TNZ), but prior to 1989, the title was the National Roads Board (NRB).

The performance model used for thin-surfaced, unbound granular flexible pavements assumes that the surface thicknesses of less than 35 mm do not contribute to the structural capacity of the pavement, and that the stresses are dissipated through the depth of the granular cover layers above the subgrade. The main criteria is the vertical compressive strain in the subgrade because the design theory presupposes that the primary mode of structural failure is permanent deformation in the subgrade. If the vertical compressive strain at the top of the subgrade exceeds the capacity of the soil, then excessive vertical plastic deformation will occur, eventually manifesting itself at the road surface as rutting. The design method is based on a mechanistic or analytical procedure, which uses linear elastic theory to calculate the stresses, strains and deformations in the pavement structure. The calculated response of the pavement is based on the subgrade strength, thickness of the different pavement layers and the compaction level of each layer.

PAVEMENT DESIGN

Flexible highway pavements in New Zealand consist of:

i) a thin surface which is most often a sprayed bituminous seal with uniform crushed stone chips, but is occasionally a bituminous mix;
ii) a basecourse which is commonly an unbound granular layer, but is sometimes a lime or cemented layer; and,
iii) a subbase which is usually one or two (and rarely more) layers of unbound granular aggregate.

Sometimes, one or more layers are stabilised or cemented to improve the properties of poorer aggregates or soils.

The flexible pavement thickness design charts in the State Highway Design and Rehabilitation Manual [1] are derived from the subgrade strain criteria for flexible pavements in the Shell Pavement Design Manual [2], with some adjustments based on research and local empirical data. The subgrade strain criterion for primary highways in New Zealand is:

$$\epsilon_{CVS} = 0.021\ N^{-0.23} \qquad \text{EQ(1)}$$

where ϵ_{CVS} is the vertical compressive strain in the subgrade, and N is the number of repeated equivalent design axle loads.

For comparison, the AustRoads subgrade strain criterion [3] is:

$$\epsilon_{CVS} = 0.0085\ N^{-0.14} \qquad \text{EQ(2)}$$

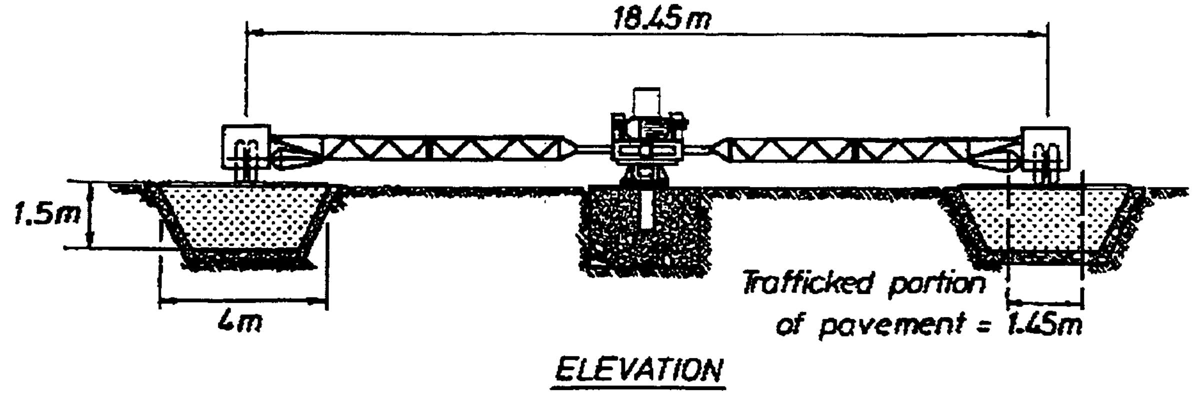

Figure 1. Simulated Loading and Vehicle Emulator (SLAVE) and Cross-section of Track

Even though the design of pavements consisting of unbound materials is based primarily on one criteria (to limit the vertical compressive strain at the top of the subgrade), other possible modes of primary failure (such as a shear failure in the basecourse) are prevented by specifying the properties of the materials. For example, the unbound basecourse aggregate has a specified gradation envelope to ensure proper compaction and effective drainage, and other properties are specified in order to avoid material degradation. Unbound granular pavements consisting of compacted well-graded crushed aggregate can sustain large numbers (over 1.5 x 10^6) of 80 kN axle load repetitions in the absence of deleterious ground moisture and environmental factors [4].

DESCRIPTION OF THE CANTERBURY ACCELERATED PAVEMENT TESTING FACILITY (CAPTIF) AND INSTRUMENTATION

The main feature of CAPTIF is the Simulated Loading and Vehicle Emulator (SLAVE), illustrated in Figure 1; the primary characteristics are summarised in Table 1. The SLAVE "vehicles" are equipped with half-axle assemblies that can carry either single- or dual-tires. The configuration of each vehicle, with respect to suspensions, wheel loads, tire types and tire numbers, can be identical or different, for simultaneous testing of different load characteristics.

Electronic systems have been developed to measure subsurface strains and temperatures, transverse and longitudinal surface profiles, and pavement rebound. The CAPTIF Deflectometer measures the surface rebound of a pavement under the influence of a wheel load, to the nearest 0.01 mm every 50 mm of horizontal movement, in much the same way as a Benkelman Beam except that the former uses an electromagnetic gap measuring sensor at the end of the beam to measure the vertical distance between the sensor and a target disc placed on the pavement surface. The CAPTIF Profilometer measures the transverse surface profiles at a transverse spacing of 25 mm, to an accuracy of ± 1 mm.

The soil strain measuring systems determine microstrains with a resolution of ±50 µm/m using Bison Soil Strain sensors. The gauge length is the separation distance between each paired coil; strain (ϵ) is the quotient of the change in gauge length (ΔL) divided by the initial gauge length (L). The strain discs are installed during the formation of the subgrade and the overlying pavement layers, resulting in negligible disturbance to the materials. Previous work which describes the use of the Bison Soil Strain Coils is given in [5].

Two data-acquisition systems for the Bison strain coils were used. In the interim system, all the coils in an array were connected to a manual switching apparatus. Then, the two leads from the switching apparatus were connected to a Bison Soil Strain Gauge, Model 4101A, a single-channel linearising monitor that supplied the alternating current to the transmitter coil and measured the output from the paired coil. The output voltages from the gauge and pavement temperature probes were recorded through the HP Model 3852A Data-acquisition Unit using a HP 44711A 24 channel FET multiplexor module. The output from the multiplexor was fed into a 13 bit digital voltmeter (HP44702A). The data-acquisition unit stored the data in memory before downloading it to a HP PC308 controller board installed in an IBM-compatible AT. The error in the interim strain-measuring system was ± 50 µm/m. Before installation, the sensors were calibrated to generate an output voltage versus separation distance relationship for each sensor configuration. During the loading routine, the normal sampling rate was 100 Hz, but the sampling rate increased to at least 10 kHz (depending on the vehicle speed) for a 0.5 second period whenever triggered by the test vehicle cutting an infra-red beam at the start of the instrumented pavement section.

The permanent system is a modified prototype of the more sophisticated Saskatchewan Soil Strain/Displacement-measuring (SSSD) system, developed by Saskatchewan Highways and Transportation, Canada. The SSSD is essentially a computer and associated units containing custom-built control, general purpose input/output, transmitter and receiver boards. Once triggered by the moving vehicles cutting a light beam, all the sensors in an array are scanned simultaneously every 30 mm of vehicle travel, and a continuous bowl of strain/displacement versus distance travelled is obtained.

The dynamic loads were quantified by accelerometers mounted at difference locations on each of the vehicles. The accelerometers, PCB 308B, are a piezoelectric type with a linear response from 1-3000 Hz of 100 mV/g. The accelerometers were connected to a PCB 483A 12 channel signal conditioning unit, which supplied power to the accelerometers and extracted the signal for output. The signals were recorded on a Hewlett-Packard 3968A instrumentation recorder, which is an eight-track frequency-modulation (FM) tape recorder. The analogue signals were digitised using the HP 3852A data acquisition system sampling at 200 Hz per channel.

TESTS CONDUCTED AT CAPTIF

The experiment involved constructing three sequential test pavements at CAPTIF (Stages I, II and III). In Stage I, the pavement response to the primary loading variables (load magnitude, tire inflation pressure and basic tire type, all on dual tired wheels) were measured. The final two stages involved testing the life-cycle performance of different pavements and subgrades under selected loading conditions.

STAGE I - PAVEMENT RESPONSE TO DIFFERENT LOADING CONDITIONS

Initial tasks included selecting and developing instrumentation and data-acquisition systems, and preparing the vehicles and track. The silty clay subgrade had a CBR of 5% at its natural moisture content. The liquid limit and plasticity index were 43 and 23, respectively. The basecourse aggregate was a well-graded, crushed rock (Canterbury greywacke), with a laboratory CBR of >100%. The moisture content during compaction was 5%. The surfacing was an asphaltic concrete consisting of a well-graded aggregate (maximum particle size 10 mm), 6.4% bitumen and 4.7% air voids. The penetration grade of the bitumen was 80/100. The bulk density and bulk specific gravity of the mix were 2300 kg/m^3 and 2.306, respectively, and the mix temperature was 155°C. The Marshall Stability and Flow were 18.1 kN and 4.3 mm, respectively.

The bottom layer in the track consisted of a 150 mm thick drainage layer of coarse gravel covered with a non-woven geotextile. On top of that was a 800 mm thick layer of well-compacted loess. The bottom coil of each vertical strain sensor array was carefully positioned in the top of the loess. The subgrade layer of silty clay was placed and compacted to a 200 mm depth over the loess. All layers of the subgrade and the granular cover were spread by a small bulldozer and compacted by a Sakai SW41 (40 kN) dual drum roller.

The next set of strain sensors and a set of temperature probes were placed at the interface of the subgrade and granular cover layers. Then, a spunbonded polypropylene geotextile was placed over the subgrade so as to facilitate the determination of subsurface layer profiles following the testing routine. The granular cover was constructed in three 100 mm lifts, to aid compaction and to allow installation of the paired strain sensors at 100 mm gauge lengths without disturbing the materials. Each array of strain sensors was installed as shown in Figure 2. Temperature probes were placed at two levels in the granular cover, beside the strain coils.

In uppermost 50 mm of the crushed rock basecourse, the maximum particle size was only 20 mm so as to aid workability and compaction. The surface of the base was finished with a pneumatic-tired roller, then a tack coat of emulsified bitumen (60%, 180/200 penetration grade) was sprayed. The asphaltic concrete was placed and levelled by hand, and compacted. A 3 m straight-edge beam was used to check the roughness, and the maximum deviation was 4 mm. The thickness of the surfacing was 35 mm (± 3 mm).

The dynamic characteristics of each vehicle were evaluated to confirm that they were similar. Dynamic wheel forces are dominated by behaviour of sprung mass, so the wheel forces are the sum of the vehicle mass multiplied by the chassis acceleration and the unsprung mass multiplied by the vertical axle acceleration. Dynamic wheel forces were measured at a constant speed of 40 km/h. The vehicles exhibited similar dynamic characteristics, and one aspect, the dynamic load coefficient (dlc), is presented in Table 2. The OECD have reported that dlc's in the range of 0.1 -0.3 have been measured under normal operating conditions [6]. Further analysis of the wheel force data has shown that the dynamic behaviour of the SLAVE vehicles is similar to measured heavy vehicle behaviour [7].

Table 1. Characteristics of the Simulated Loading and Vehicle Emulator (SLAVE)

Item	Characteristic
Test Wheels	Dual- or single-tires; standard or wide-base; bias or radial ply; tube or tubeless; maximum overall tire 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 wheelpaths
Speed	0-50 km/h, programmable, accurate to 1 km/h
Radius of Travel	9.1 m

Table 2. Dynamic Load Co-efficients (dlc)

Vehicle	Wheel Load (kN)	Tire Pressure (kPa)	Tire Type	dlc
A	40	580	Bias ply	0.22-0.24
B	40	580	Radial ply	0.22-0.24
B	46	825	Radial ply	0.16-0.18

Dynamic load co-efficient (dlc) =

$$\frac{\text{Std Deviation of Wheel Forces}}{\text{Static Load}}$$

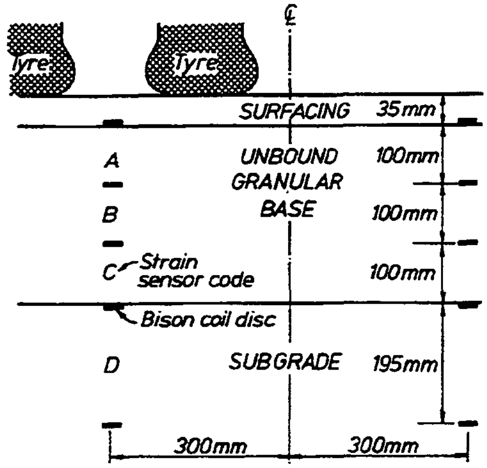

Figure 2. Array of Strain Coil Sensors

Vehicle A carried a constant half-axle load of 40 kN (equating to a full axle load of 80 kN) with dual bias ply tires inflated to 550 kPa, so that it was a reference throughout the testing routine. The characteristics of vehicle B were modified. The tires were inflated to the maximum cold pressures allowed by the tire supplier (700 and 825 kPa for the bias and radial ply, respectively). All radial and bias ply tires were 10.00R20 and 10.00x20, respectively. The experimental matrix of loading conditions is shown in Table 3.

The surface deflection bowls, and the vertical strains at various depths in the pavement and subgrade were measured for each of the 20 loading conditions. The strains were measured under the centre of the dual-tires. Longitudinal and transverse profiles were measured after specified increments of cumulative loading cycles.

Table 3. Sequence of Loading Conditions on Vehicle B.

Bias ply Tires Test No.	Pressure (kPa)	Wheel Load (kN)	Radial ply Tires Test No.	Pressure (kPa)	Wheel Load (kN)
1	550	40	9	825	46
2	700	40	10	700	46
3	700	21	11	550	46
4	550	21	12	550	31
5	550	31	13	825	31
6	700	31	14	700	31
7	700	46	15	700	21
8	550	46	16	550	21
			17	825	21
			18	825	40
			19	700	40
			20	550	40

After the experimental matrix was completed, the maximum surface rut depth was only 7 mm. Most of the deformation occurred in the subgrade, and the deformation within the unbound granular pavement was only 2-3 mm. Therefore, the permanent deformation was relatively small during the testing routine, and is not considered to have substantially affected the consolidation and the properties of the layers.

Deflection basins were measured at three locations on the test section, and averaged to determine one basin for each experimental point. The deflection basins were compared on the basis of (a) tire type, (b) tire inflation pressure, and (c) wheel load; neither tire type nor tire inflation pressure had a substantial affect on the deflection basin shape. In general, the peak deflection value was independent of tire type. Also, tire pressure tended to have a negligible affect on peak value. The major influence on peak deflection was the wheel load.

The vertical compressive strains were measured in three layers (upper basecourse, lower base and subgrade) for each of the 20 loading conditions. The peak compressive strains are defined as the difference between the nominal average residual strain recorded prior to the approach of the test vehicle to the sensor and the maximum strain (averaged over five cycles) measured under the test vehicle. Figure 3 shows a representative sample of the original data from the subgrade, for one specific loading condition (dual radial tires, inflated to 825 kPa and loaded to 40 kN). The longest spikes represent the passage of a wheel load directly over the sensors, and the lessor spike is the passage of the reference wheel load (dual bias ply type inflated to 550 kPa and loaded to 40 kN) at a transverse distance of .6 m. For every loading condition and strain measurement in the subgrade, some vertical compression remained after the passage of each wheel load. Then, when the other wheel load passed over the station, the compression disappeared and the subgrade reverted to the original situation. This phenomena repeated itself for all loading cycles.

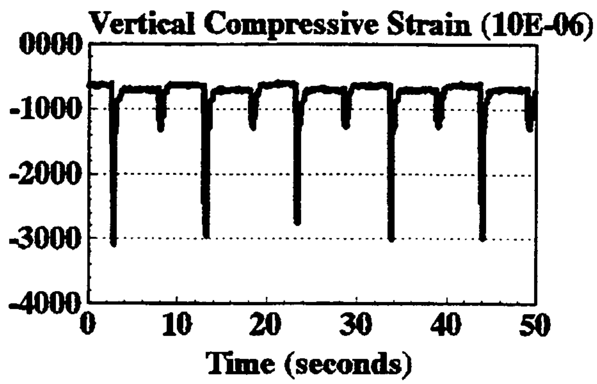

Figure 3. Original strain data from the subgrade in Stage I.

If it was simply a case of the sensors' axes becoming misaligned, then the effect would have increased with cumulative cycles, but this did not occur. Instead, the subgrade was compressed when one wheel load passed directly over the sensor, then shear forces created by the other wheel load, travelling in a wheel path 0.6 m away laterally, resulted in extension in the layer, and the sensor returned to its original position. Similarly, in the basecourse, the residual compression induced by one vehicle passing directly over the senors was eliminated by extension as the other vehicle passed over a point 0.6 m away transversely, but there was no discernible resilient compression as the second vehicle passed over, as shown in Figure 4. This cyclic compression and extension contributed little to the permanent deformation of the layers, which is the primary criterion for the model describing the performance of thin-surfaced unbound granular pavements, but could affect the degradation of the materials.

The magnitude of the vertical compressive strain in the subgrade and the basecourse (Figures 5 and 6 respectively) actually decreased as the tire inflation pressure increased, for every wheel load. The vertical compressive strain in the granular layers and the subgrade must be dependent upon the zone of influence of the load, as well as the contact area and speed of the vehicle. Thus, when the speed is constant and the contact area is reduced, at higher tire inflation pressures, the zone of influence of the load in the pavement and subgrade is reduced, thereby reducing the strain induced in the subgrade.

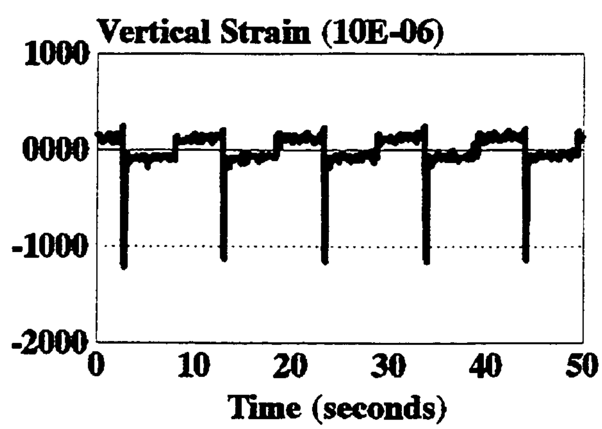

Figure 4. Original data from the basecourse in Stage I.

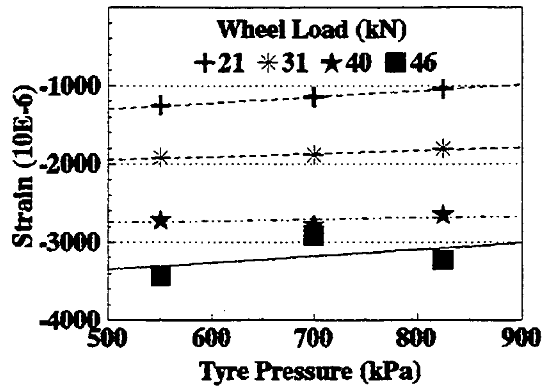

Figure 5. Subgrade strains compared to tire inflation pressure and axle load in Stage I.

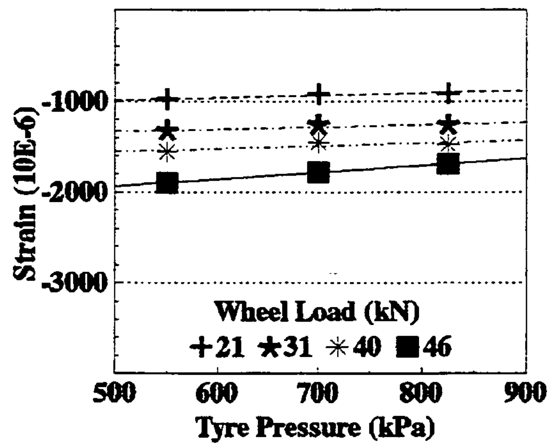

Figure 6. Basecourse strains compared to tire inflation pressure and axle load in Stage I.

STAGE II - PEFORMANCE OF AN UNBOUND GRANULAR BASECOURSE UNDER A THICK ASPHALT SURFACE

The Stage II pavement had an asphaltic concrete surfacing 85 mm in thickness (which is thick by New Zealand standards), over 200 mm unbound granular basecourse over a clayey subgrade. The *in situ* CBR of the subgrade for the test section was 13%. The basecourse aggregate was a well-graded, crushed river gravel, compacted with a moisture content of 4% to a maximum dry density of 2150 kg/m^3. The asphaltic binder was a plastomer-modified bitumen called Practiplast, with a penetration grade of 50 (@ 25° C), a softening point of 59.4 °C, a viscosity of 2 P @ 169°C, and a shear susceptibility of -0.089 [8].

The wheel load was 40 kN for both vehicles for the first 920,000 loading cycles, and 46 kN for the remaining 1.2 million loading cycles. The dual radial tires in both vehicles were inflated to 700 kPa, and the vehicle speed was 40 km/h for routine loading, and 20 km/h for the approximately 500 cycles required for completing the strain measurements. The vertical compressive elastic strains in the unbound granular basecourse and clayey subgrade were measured using arrays of Bison strain coil gauges installed in the subgrade and basecourse. Altogether, SLAVE applied over 3.2 x 10^6 EDA to the test pavement.

The rut depth at the surface of the test section was only 2-4 mm, indicating negligible deformation in the subsurface layers. The project concluded before the pre-defined failure criterion of a maximum surface rut depth of 25 mm occurred because the pavement design was conservative and the maximum allowable expenditure on the project was reached. Surface cracking was insignificant.

In the basecourse, the magnitude of the peak vertical compressive strain decreased slightly during the cumulative loading, from 300 to 220 µm/m. The relationship between the magnitude of the vertical compressive subgrade strain and cumulative loading is shown in Figure 7. The temperatures are the averaged output of the three temperature probes installed directly above the strain sensors in the 85 mm thick asphaltic concrete. Increasing the wheel load from 40 kN to 46 kN, after 920,000 EDA, resulted in a negligible change in the magnitude of the vertical compressive strain responses in the subgrade (from 1200 to 1250 µm/m) and no change in the basecourse

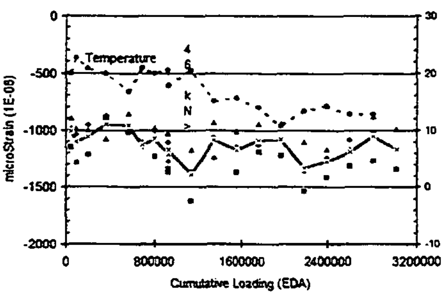

Figure 7. Subgrade strains in Stage II

strain. In Stage I of this research program, the same increase in wheel load produced a 10% increase in the magnitude of the vertical compressive strain in a weak subgrade (CBR of 4%) under a thin-surfaced, 300 mm unbound granular pavement.

The nominal magnitude of the vertical compressive strain in the subgrade varied between 900 and 1400 µm/m, and the pavement survived 3.2 x 10^6 EDA without incurring any substantial permanent deformation in the pavement or subgrade. When the temperature of the asphaltic concrete decreased, thereby increasing the asphalt modulus, the subgrade strain decreased because the stiffer asphalt is more effective in dissipating the stresses from the wheel load. The effect of speed (between 20 and 40 km/h) on vertical compressive strain in the subgrade was negligible.

Employing equations (1) and (2), the maximum allowable vertical compressive strain in the subgrade would have been 670 µm/m and 1045 µm/m, respectively, for that number of loading cycles, assuming that the pavement would have failed at that point. The pavement design procedure was definitely conservative, but the AustRoads criterion permits significantly higher strains, which are within the range of the actual strain values measured in this case. Also, the lack of adverse environmental effects would have contributed to extending the life of the pavement.

STAGE III - LIFE-CYCLE PERFORMANCE OF A THIN-SURFACED UNBOUND GRANULAR PAVEMENT

The test pavement for this trial consisted of 25 mm asphaltic concrete surfacing layer over 135 mm thick basecourse of unbound granular aggregate on a silty clay subgrade of CBR 13%; the material properties are the same as those described above. The pavement responses and properties were measured as described above. The pavement was subjected to a constant loading condition (40 kN load and dual radial tires inflated to 825 kPa) until the loading concluded at 740,000 EDA, when the permanent vertical deformation (rut depth) of the surface reached 28 mm (the definition of failure was a maximum rut depth of 25 mm).

The maximum rut depths over the whole pavement were in the range of 15 to 28 mm. In the excavated trenches, the asphaltic concrete surfacing and basecourse of unbound aggregate compacted 8 mm and 7 mm, respectively, at the centreline under the cumulative loading, while the permanent deformation in the subgrade varied between 1 and 13 mm. Most (75%) of the permanent deformation occurred in the first 100,000 EDA, then the rutting progressed at a relatively constant rate of 9 µm/ loading cycle (1 loading cycle equals 2 EDA). The only significant difference in the longitudinal surface profiles before loading commenced and at the end of the trials was where a localised failure was repaired.

The peak surface deflection was approximately 1.6 mm throughout the life of the pavement, except for a temporary 0.15 mm increase in surface deflection that occurred between 100,000 and 200,000 cumulative EDA. The deflection bowl shapes were the same temporally as well, indicating that the relative moduli of the various layers did not change.

Figure 8 illustrates how, after the initial sharp increase in magnitude in the peak vertical compressive strain in the subgrade (ϵ_{cvs}), the strain under cumulative loading varied little, except for the significant decrease at 220,000 EDA. The magnitude of the peak vertical compressive strain in the basecourse slowly decreased during the first 300,000 loading cycles, from 3200 to 2350 µm/m, then remained relatively constant, as shown in Figure 9. The effect of speed (between 20 and 45 km/h) on the vertical strain at each of three stations varied slightly, though, on average, the effect of speed on vertical compressive strain induced in the subgrade was negligible.

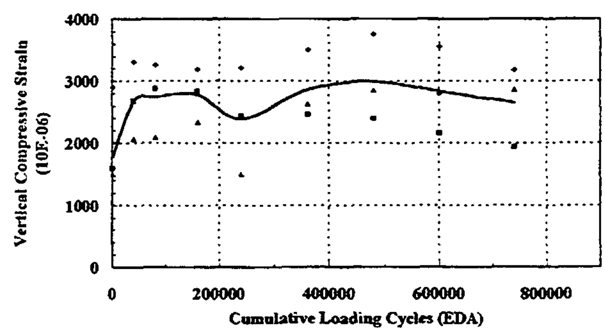

Figure 8. Subgrade strains in Stage III.

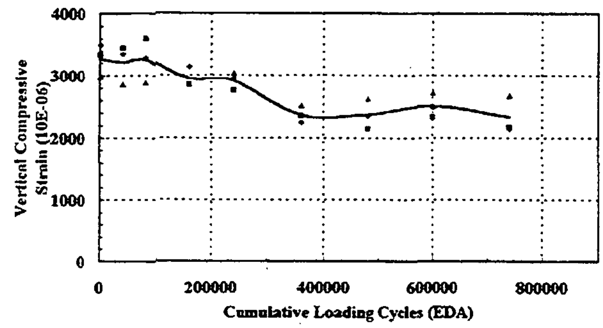

Figure 9. Basecourse strains in Stage III.

Until approximately 300,000 EDA,surface deflections and vertical compressive strain levels in the basecourse and subgrade layers fluctuated. The basecourse strain levels tended to decrease in magnitude, while the magnitude of the subgrade strain tended to increase, until the pavement and subgrade responses achieved a stable condition, with only minor fluctuations in the response to load.

Using equations (1) and (2), the maximum allowable vertical compressive strains in the subgrade for 740,000 EDA are 940 μm/m and 1280 μm/m, which are substantially less than the actual strains of approximately 2800 μm/m. Table 4 shows that the actual strains are two to three times the theoretical maximum strain magnitude allowed by the different criterion, which suggests that the criteria on which the pavement thickness design charts are based could be conservative. However, as before, the AustRoads value is much closer to the measured strains.

Relating Pavement Response to Performance

The objectives of Stages II and III were to use the system developed in Stage I to relate axle loads directly to pavement responses for predicting pavement performance.

Stage II was actually conducted simultaneously with another project investigating the effect of different modified binders on the performance of asphaltic concrete surface layers. Stock *et al* [8] concluded that the thinner asphaltic concrete layers constructed with modified binders and the high-stiffness binder provided performance equivalent to that of the thicker layer containing a conventional binder, and that the design procedure was conservative because pavements designed for 1 x 10^6 EDA should have exhibited greater deterioration after 3.2 x 10^6 EDA.

FIELD MEASUREMENT OF PAVEMENT STRAINS

The pavement response was measured using the interim Bison strain measuring system as described above. The portable data-acquisition equipment was completely self-sufficient. The field work was conducted in January 1992 [9]. Ten pairs of Bison coils were placed in a trench in an existing pavement in five arrays of two pairs each, as shown in Figure 10. The road had been designed to carry 7x10^6 EDA over a fifteen year design life. The pavement consisted of a two coat chipseal on top of 300 mm of unbound granular basecourse. The basecourse was placed on the subgrade which had a CBR of 23%.

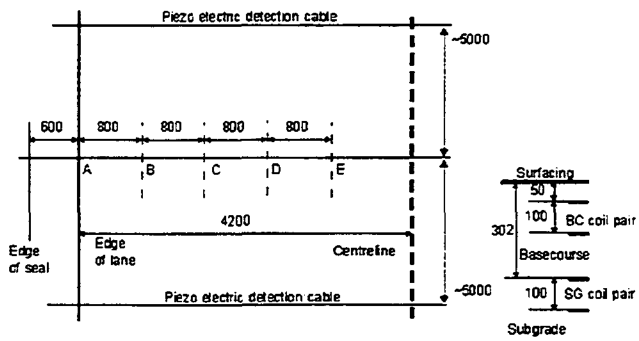

Figure 10. Layout of instrumentation for the field trials.

The section of road selected for this investigation showed no signs of structural distress. The compacted density of the subgrade was 1753 kg/m^3.

Four series of tests at different speeds (5, 20, 40 and 60 km/h) were undertaken using a Bedford truck with a single tired front axle and a driven rear axle fitted with dual 10.00 R 20 radial ply tires. The truck was loaded so that the rear axle applied a standard load of 8.2 tonnes to the pavement. The tires were inflated to 580 kPa.

Nine series of tests were performed by an A-Train logging truck. The test combinations were a matrix of three speeds (5, 20 and 60 km/h) and three axle weights (8, 12 and 16 tonnes). All of the truck axles were dual tired (13.00 R 22.5 tires, inflated to 689 kPa), with the exception of the steering axle, which was a single tire axle. Logs were used to provide the required weight

Table 4. Allowable vertical compressive strain models compared with actual values.

Pavement Number	Number of Load Repetitions (EDA)	Maximum Allowable Vertical Compressive Strain (μm/m) in the Subgrade: New Zealand[a]	AustRoads[b]	Actual (Nominal Value)
1[c]	3200000	670	1045	1200
2[d]	740000	940	1280	2800

[a] Equation (1)
[b] Equation (2)
[c] 85 mm asphalt surface, 200 mm unbound granular basecourse, silty clay subgrade CBR 13
[d] 25 mm asphalt surface, 135 mm unbound granular basecourse, silty clay subgrade CBR 10

for each test combination. The trailer was fitted with a basic walking beam suspension, with no damping, which meant that the gross weight of a trailer was supported equally on all four axles (the two rear axles of the tractor unit and two axles at the rear of the trailer). The tandem axles were spaced at 1.5 m centres, and the distance between the centres of the tandem axle groups was 8.6 m.

ANALYSIS OF FIELD RESULTS

For each test combination and coil pair, at least ten strain values were averaged to produce a single strain value for each test condition and coil pair. The variation of values in each data set was approximately 5%, so the testing was reproducible enough to be able to exclude any outlying values.

The purpose of this investigation was to measure the peak vertical compressive strain in the pavement, so fitting a regression line to the maximum strain values at each data point was the most representative interpretation of the results.

The results showed that the peak vertical compressive strain was related to axle weight on a linear basis, in both the subgrade and the basecourse, as shown in Figure 11. Analysis of the strain versus vehicle speed showed that the strain in the basecourse decreased as the speed increased, which confirmed the findings of Brown and Pell [10], as shown in Figure 12. However, the strain in the subgrade increased as the vehicle speed increased, which contradicted the findings of Brown and Pell [10].

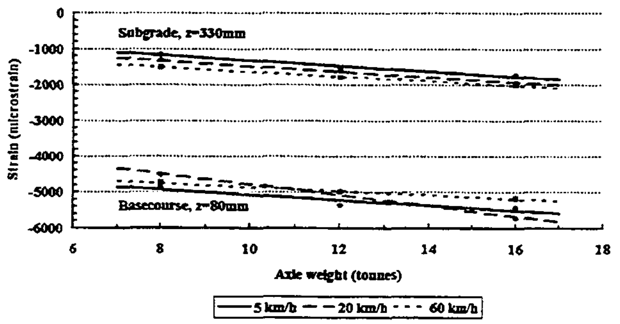

Figure 11. Pavement strains compared to axle weights in field trials.

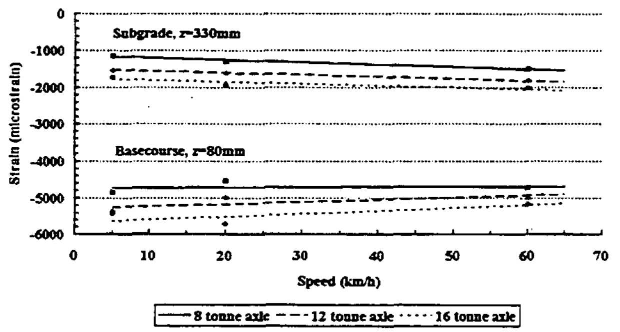

Figure 12. Pavement strains compared to vehicle speed in the field trials.

When the single and tandem dual tired axle results were compared, the strains in the basecourse induced by a single axle from a tandem axle group were 32% greater than the strain induced by a single axle, at all of the test speeds. The subgrade exhibited a variable response under loading by the two different axle configurations. At a test speed of 5 km/h, the tandem axle group induced a strain that was 24% greater than the single axle strain. At test speeds of 20 and 60 km/h, there was less than 4% difference between the strains induced by the different axle configurations. The variation in differences may be due to the difference in the suspension types; multi leaf steel springs were on the single axle and walking beam were on the tandem axle group. Another reason could be the difference in tire sizes.

CONCLUSION

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

With respect to pavement and subgrade response to loading, and for the specific conditions of the investigation, the tire type (10.00R20 radial and 10.00x20 bias ply) had an insignificant effect, and the tire inflation pressure (between 550 kPa and 825 kPa) had a minor effect. The vertical compressive strain in the subgrade and unbound granular layers of the pavement actually decreased as the tire pressure increased.

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

Trials involving instrumented pavements in service and in the Canterbury Accelerated Pavement Testing Facility have shown that actual vertical compressive strains in the top of the subgrade are up to three times the theoretical maximum allowable, depending on the criterion used, which suggests that the criterion on which the pavement thickness design charts are based could be conservative. However, the strains predicted by the AustRoads model (Eqn. 2) are closer to the measured strains.

In the field trials, the pavement response showed a linear change with respect to either the axle weight or the vehicle speed. Again, as shown in the test at CAPTIF, the actaul measured strain levels were up to three times the value predicted by the New Zealand pavement performance model. However the instrumented road was still showing an acceptable level of service relative to the age of the pavement structure.

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