Shakedown Analysis of Road Pavements - An Experimental Point of View

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Summary: Shakedown behaviour of road pavements is investigated in laboratory controlled conditions using the Sydney University Pavement Testing Facility. The Pavement Test Facility was upgraded to provide improved data acquisition, storage and analysis for the experiment. Wheel loads lower than the shakedown load generated low permanent deformations for a larger number of load cycles in comparison with high permanent deformations for a lower number of load cycles for wheel loads higher than the shakedown load.

INTRODUCTION

Pavement design decisions are primarily based on the pavement life cycle cost together with an acceptable pavement maintenance regime for the particular road in consideration. Functional failure (i.e. increase in surface roughness) is an important factor in the design process although the pavement needs to be designed against structural failure in order to carry the design loads. The design inputs range from axle load and traffic analysis, environment factors, pavement material properties, improvement of locally available material, properties of the subgrade, properties of bases and sub bases, available construction standards and equipment, surfacing material and fundamental stress-strain analysis. The pavement design process can be considered from two broad points of view. Performance based pavement design processes, that extrapolate data from various trials and other prototypes are favoured by practicing engineers. Results of WASHO Road Tests in Idaho (1952 – 54), AASHO Road Tests in Illinois (1958–60) and performance observations carried out on existing road sections in the UK by TRRL from Boroughbridge (1949) to Conington, Cambridgeshire (1965) have had a major influence on contemporary design concepts. Current "mechanistic" pavement design procedures are based upon a failure mechanism identified with certain critical elastic strains reaching a critical level, whereas embankments and foundations are designed against failure by using plastic analysis. Pavement design procedures such as LR 1132 (1984), AASHTO (1994) and AUSTROAD (2000) are based on an elastic stress/strain relationship.

An analysis which incorporates the substantial strength existing prior to the point of static collapse has been suggested by (Sharp and Booker, 1984) who pioneered the application of Melan's (1936) Shakedown Theory to road pavements. The shakedown model identified a critical load level below which shakedown occurs, but above which permanent strains continue to occur. Extensions for Melan's lower bound approach to calculate the shakedown load was presented by (Raad et al 1988, 1989), (Hossain and Yu, 1996) and (Shiau and Yu, 2000). Upper estimates of the shakedown load were obtained by (Collins and Cliffe, 1987) employing the dual kinematic theorem due to Koiter (1960). They have shown that in the two dimensional case results were identical with Sharp and Booker's lower bound approach. Sharp and Booker, (1984) have shown AASHTO (1950) experimental results agree with their parametric study, they concluded that there is a clear need for further experimental work so that such design criteria could be validated. In this paper, it is intended to discuss a laboratory experiment presently underway at the University of Sydney Pavement Testing Facility in Australia, to examine the application of shakedown theory by means of measuring accumulated plastic deformation, after applying traffic loads below and above the theoretical shakedown load. To facilitate the data acquisition, storage and analysis, a new data acquisition system was developed and installed for the Pavement Test Facility as a part of this experiment.

SYDNEY UNIVERSITY PAVEMENT TESTING FACILITY

Introduction

The testing facility was initially developed by Wong and Small (1994) to test model pavements and was modified in order to change the position of the tyre across the pavement randomly in Moghaddas-Nejad and Small (1996). The facility consists of three main structural components, namely, the test tank, the overhead track and the loading carriage. The support and guidance for the moving loading carriage is provided by the overhead rails. The test section of pavement is constructed inside the test tank and positioned below one of the straight sections of the overhead track. The test wheel runs on a plywood track when outside of the test section of pavement. A conductor rail system supplies power to the motor which drives the wheel. The wheel passes over the test section of pavement once during each revolution around the track and triggers a micro switch that starts a micro computer recording data. Test facility specifications are shown in Table 1.

Table 1. Test facility specifications.

Main Features of the Pavement Testing Facility	
Feature	Specification
Speed range (km/h)	0-7.2
Wheel load (kN)	0-1.4
Maximum tyre pressure (kPa)	500
Tyre width (mm)	45
Tyre diameter (mm)	220
Length of test section (m)	1.4
Width of test section (m)	0.5
Maximum depth of tank (m)	0.8
Length of test track(m)	12.15
Time to complete one circuit at 1 km/h (s)	44.0

Instrumentation and Data Acquisition

All subsurface settlements are measured by buried Perspex discs (30 mm diameter) that are connected to a wire that passes through the base of the tank and is connected to a transducer. Data acquisition from transducer output is carried out only during the passage of the wheel across the test section of pavement.

Loading Wheel

The loading wheel is supported by a rotating arm which is pivoted at the top by means of roller bearings. The position of the spring loading system is almost in the middle and the loading wheel is at the end of the horizontal portion of the rotating arm. By means of a hinged connection, the spring loading system is connected to the bottom bogie plate as indicated in Figure 1.

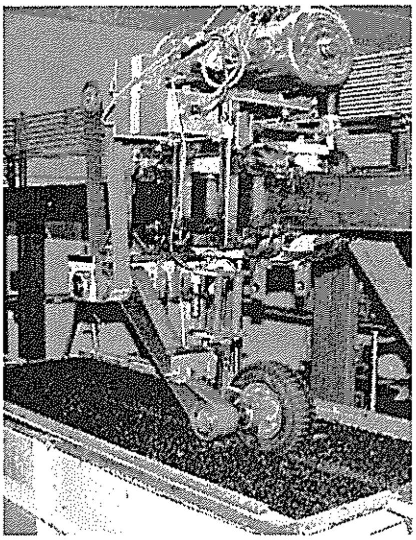


Figure 1. Spring loading arrangement and the loading wheel.

To adjust the spring load on the rotating arm, a set of two adjusting nuts has been provided above and below the rotating arm so that compression in the springs can be changed by adjustment of the upper nut. Four identical springs with guide rods placed concentrically inside them make up the loading unit. To satisfy different loading requirements, springs with different stiffnesses can be set up in the system. A LVDT transducer fitted between two reaction plates is used to monitor the variation of compression in the springs. There are two aluminium boxes supported by the bogie connection frame. The smaller box contains a power supply for the transducer and the larger one contains the analog to digital signal converter, for transmission of data from the wheel carriage.

Measurement of surface settlement

To cater for the large number of cross section measurements envisaged in the project, a new laser transducer based surface deformation measurement system was designed and developed to measure and record the surface deformation measurements. This system includes a cable mounted slider with a key hole and a rail fixed to a detachable arm with the laser transducer as shown in Figure 2.. The position control system is based on a relay controlled small DC motor and a pot resistor.

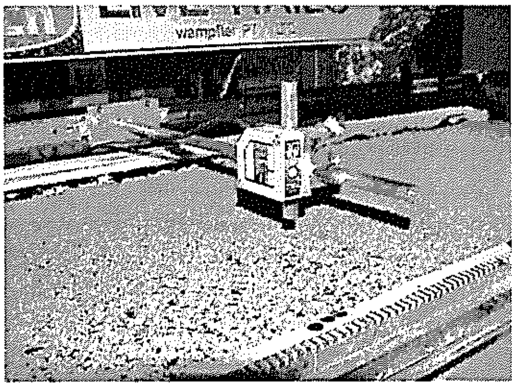


Figure 2. New cross section measuring arrangement using a laser.

All measurements were recorded directly to a database. Graphical user interface based software was developed to plot cross sections and a Visual Basic AutoCAD application was developed to plot a 3D grid depicting the surface deformations. This enabled plotting of accurate surface deformation patterns to study the pavement shakedown.

MATERIAL PROPERTIES

Sand Subgrade

Loose Silica Sand was used as the subgrade. Particle size distribution is as shown in Table 2.

Table 2. Particle size distribution of subgrade sand (Test Specification AS 1289 3.6.1).

Sieve size (mm)	4.75	2.36	1.18	0.600	0.425	0.300	0.15
%Passing	100	99	99	98	89	54	2

Base Material

Base material used in the experiment was made up of recycled crushed concrete aggregate obtained from Randwick Yarra Bay Material stockpile site. This material mainly consisted of crushed concrete obtained from council building demolition sites. This material is used as a base material in road rehabilitation works in the City Council area and is found to be easy to handle at the site with respect to compaction and spreading.

All material tests carried out were compared with current AUSPEC specifications and relevant ARRB recommended specifications for recycled crushed concrete specifications. Direct Shear Box tests and Texas Triaxial tests were carried out to determine the angle of internal friction and cohesion values. Test results are shown in Tables 3 and 4.

Table 3. Soil strength test results for base material.

Texas Triaxial	Direct Shear Box
ϕ =46 degrees c = 45 kPa	ϕ = 51 degrees c = 56 kPa

Table 4. Particle size distribution test results for base material.

Sieve Size (mm)	Sample				ARRB Spec.
	1	2	3	4	
26.5	100	100	100	100	100
19	93	97	98	95	95 – 100
13.2	77	76	82	81	75 – 95
9.5	65	66	69	72	
6.7	58	59	61	65	
4.75	51	53	54	58	42 – 76
2.36	44	44	46	50	28 – 60
1.18	38	39	40	43	
0.6	32	33	34	37	
0.425	27	27	27	31	10 – 28
0.30	17	18	17	20	
0.150	6	7	6	8	
0.075	4	4	4	5	2 – 10

CONSTRUCTION OF PAVEMENT

Pavement

A Standard Proctor hammer was used to compact the pavement layers. Moisture content was maintained at OMC and the numbers of blows applied were evenly spread across the layer and were made equal to give the same energy level to the base material as in the Standard Compaction Test. The pavement surface was allowed to dry at 20^0 C before lightly brushing off the loose fines in order to apply the bitumen emulsion seal coat with 5 mm single sized cover aggregate. Emulsion was applied with a roller brush and spread evenly across the pavement. After application of the cover aggregate, a light compaction pass was applied to embed the cover aggregate into the emulsion layer as a single layer of thickness equal to the aggregate's least dimension. A further 24 to 48 hours drying period was allowed before brushing the loose cover aggregate off the surface. To date, three different test pavements have been tested in the experiment. Wheel entry and exit sections were kept at 350 mm thickness for all tests and the 700mm length mid section thickness was varied to obtain different theoretical shakedown loads. General pavement configuration is shown in Figure 3.

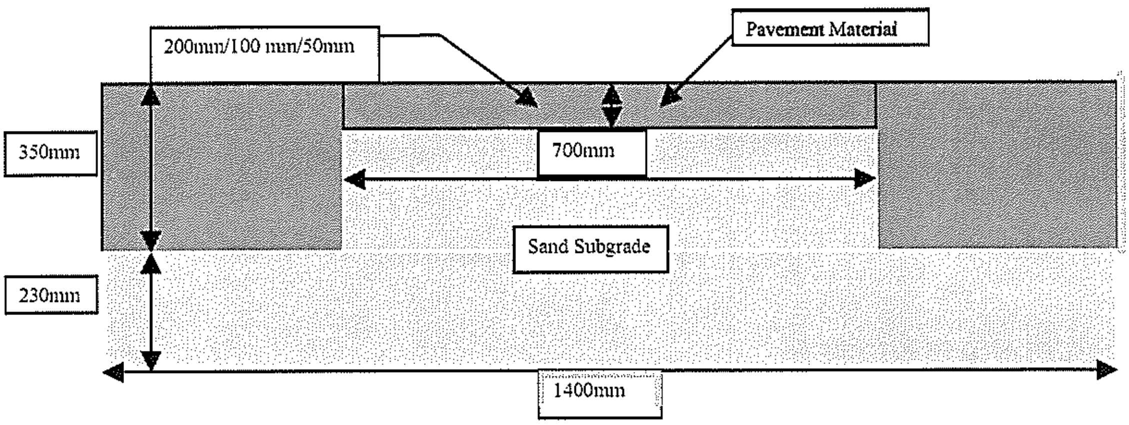


Figure 3. General pavement configuration.

CALCULATION OF SHAKEDOWN THICKNESS

General

The shakedown theorem due to Melan (1936) as presented by Maier (1969), described the main limitations of the classical theory as:

"(a) *Inviscid perfectly-plastic* (non-hardening) laws govern the local deformability, and involve *convex* yield surfaces, *associative* flow rules and *constant* elastic moduli (the term *inviscid* rules out time-effects, such as creep and rate-sensitivity); (b) geometric changes do not significantly affect the equilibrium relations; (c) temperature changes have negligible influence on the material properties; (d) external agencies act so slowly that the system behaves in a *quasi-static* way (with negligible inertia and viscous forces); (e) adaptation guarantees the survival of the structure, i.e., rules out structural crises by excessive deformation or local failure."

The question of the validity of the shakedown theorems for materials with non-associated flow rules has been examined by Maier (1973). He showed that the bounds given by Koiter's theorem are still upper bounds to the shakedown load, even though the real material has a non-associated flow rule. Melan's theorem can be used to obtain a lower bound to the shakedown limit for a non-standard material, but the yield-surface must be replaced by a "potential surface" which lies inside the yield surface.

Sharp and Booker (1984) applied the linear programming technique adopted by Maier (1969). They assumed a plane strain model with a trapezoidal pressure distribution under a roller. The material of the half space was assumed to be isotropic and homogeneous and the resulting permanent deformation and residual stress distribution were assumed independent of horizontal distance and dependent on the depth. The tangential shear load was taken as a trapezoidal distribution. The failure criterion was Mohr-Coulomb and material properties were assumed to be linear elastic – perfectly plastic.

Calculation of Shakedown Limit

Calculation of shakedown limits for the various pavement configurations tested in this experiment is based on a lower bound calculation procedure developed by S.H.Shiau (2001) in his PhD thesis. Calculation procedure is depicted in Figure 4. This procedure assumes that both elastic stresses and residual stresses required by the lower bound shakedown analysis are linearly distributed across the continua and the resulting deformation is plane strain by replacing the wheel load as an infinitely wide roller. A trapezoidal load distribution was selected as the contact load distribution. The finite element and the linear programming approach are used in this procedure.

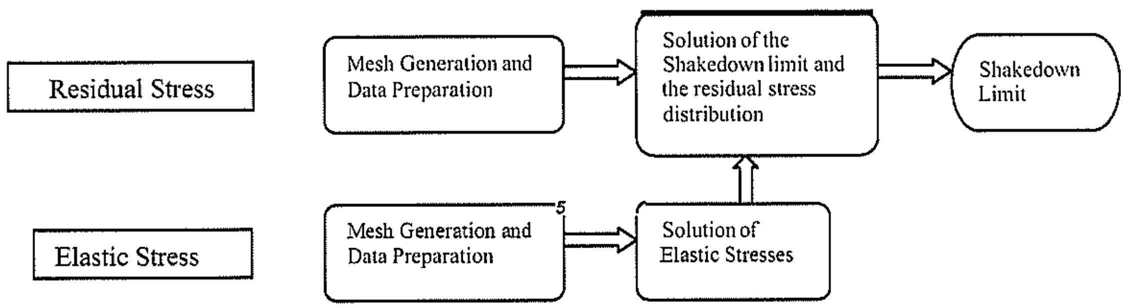


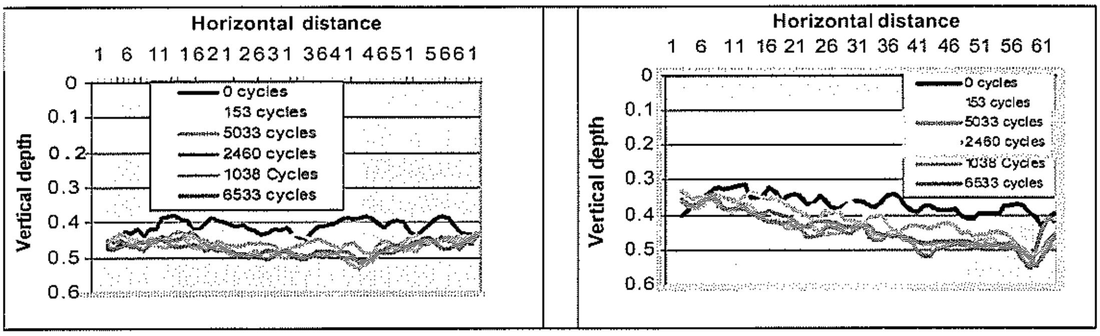
Figure 4. Calculation of shakedown limit.

TEST RESULTS AND DISCUSSION

Database

Data acquired comprised subsurface pavement settlement data from LVDTs, the time of reading, the lateral location of the wheel (as the wheel can be moved laterally relative to the pavement), number of test cycles, spring load monitoring data, subsurface permanent deformation details and transducer calibration data. Results were written to a relational database enabling online analysis and processing (OLAP) of test data.

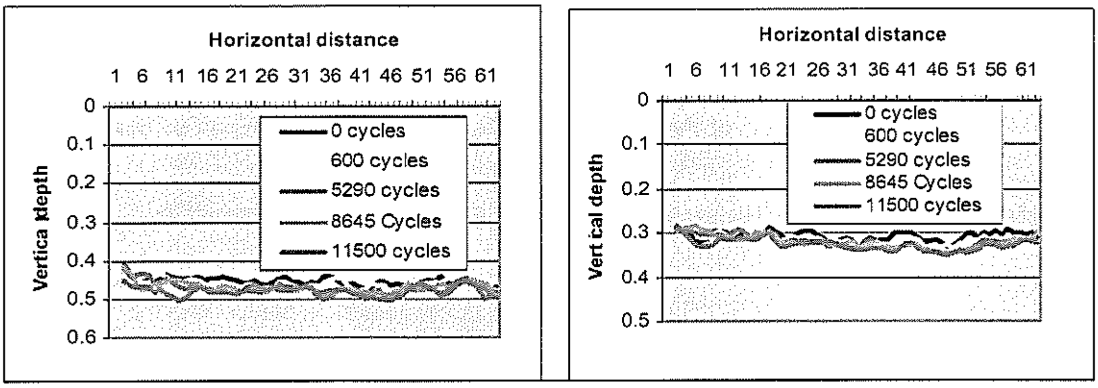
Some typical pavement settlement patterns from the cross-section measurement database are shown in Figures 5 to 6 and a deformed surface plot is shown in Figure 7. Results so far indicate that larger deformations occur for fewer load cycles when the wheel load is more than the shakedown load.



Vertical settlement pattern at 1000 mm from edge

Vertical settlement pattern at 400 mm from edge

Figure 5. Pavement settlement pattern when wheel load (SON) is more than the theoretical shakedown load (5N) of the test pavement (Pavement thickness 50 mm).



Vertical settlement pattern at 1000mm from edge

Vertical settlement pattern at 500 mm from edge

Figure 6. Pavement settlement pattern when wheel (SON) load is less than the theoretical shakedown load (229N) of the test pavement (Pavement thickness 200mm).

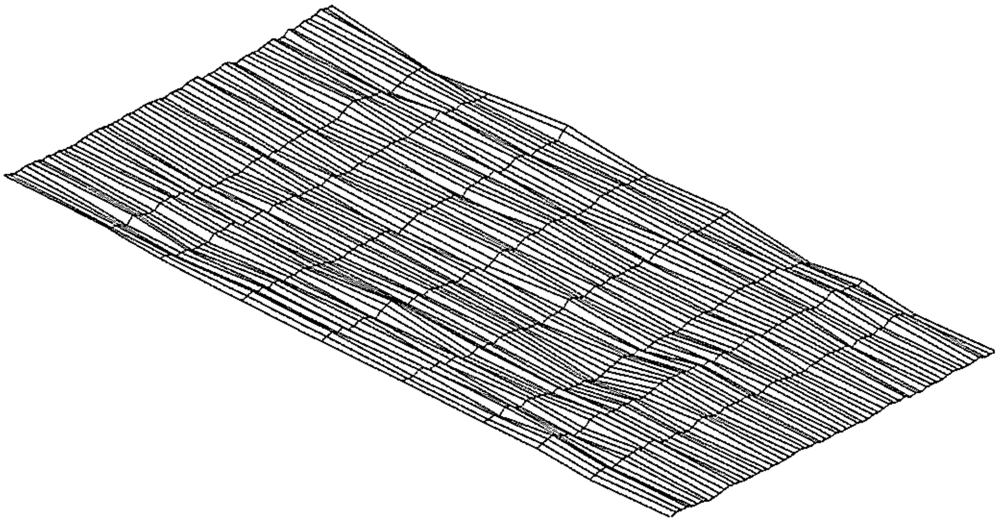


Figure 7. 3D mesh representation of pavement vertical surface deformation (VSD) plotted directly from the test results data base (after 4800 cycles when wheel load (SON) is more than shakedown load (5N)) (Pavement thickness 50 mm).

Test Results

Vertical surface deformation (VSD) of the pavement surface was measured at nine cross sections initially. The number of cross sections was increased to cover the progression of VSD and to produce 3D mesh images of the pavement surface. This made it possible to visualize the settlement pattern of the total pavement rather than individual cross sections. Standard measures such as pavement rutting and roughness are related to VSD but do not have the same degree of reliability. Usually rutting is determined by calculating or measuring the depth of a rut from a straight edge placed across the wheel path and is affected by any heaving at the edges. Roughness represents the variation of VSD along the wheel path. Results obtained by this experiment so far indicate that there is a rapid increase in VSD in the case of wheel loads more than the shakedown load calculated by the two dimensional shakedown load for the test pavement. In the current tests, trafficking was terminated when the pavement reached a shakedown state or when VSD increased more than 10 mm in a particular cross section (this corresponds to a 40 mm depth in the prototype). More tests are needed to verify the long term behavior of the pavement.

CONCLUSIONS

Preliminary testing of pavements with the University of Sydney Pavement Testing Facility have indicated that the shakedown loads predicted by 2-D shakedown theory analyses are a good indicator of whether a pavement will undergo continued deformation under cyclic wheel loading. Results so far have indicated that at loading levels above the shakedown limit of the test pavement, VSD increased rapidly to form a rut indicating pavement failure and when the loading levels were lower than the shakedown limit, after an initial vertical settlement VSD remained constant for a sufficiently large number of cycles.

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