Field and Laboratory Tests on Granular Pavements

by

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SUMMARY. The design of a granular pavement constructed on a sand subgrade at the Narrows Interchange, Perth, was based on elastic theory. Opportunity was taken to test such theories by measuring vertical stresses and deflections and comparing them with the predictions. Field and laboratory measurements of elastic modulus were obtained. It was found that the theory gave reasonable predictions of behaviour, thus giving more confidence in the use of such approaches.

1 INTRODUCTION

From observations of the performance of thin pavements (i.e. of the order of 150 mm) over sand subgrades it appears possible that pavement design methods which have been developed for plastic subgrades can produce extremely conservative designs when used in this situation. This fact creates an incentive to use a method of design which is related to the structural behaviour of the various parts of the pavement rather than on material properties which are correlated with pavement performance. One such elastic design method was published in the early sixties (Refs. 1,2).

In the Metropolitan Region of Perth, Western Australia, the subgrades are predominantly of sand. For this condition the Main Roads Department has developed a method of pavement design based on a stage construction process, the pavement being gradually upgraded as the traffic increases. The initial pavement thickness and subsequent overlay thicknesses are based on the known performance of similar roads in the region. In the case where pavements are initially constructed to their final standard, as is the case for Freeways, a more formal approach to pavement design is necessary. In the case of the pavement to be discussed here the final thickness was fixed using the Shell Design Charts (Ref. 2).

The pavement is part of the Narrows Interchange complex of the Mitchell Freeway and is situated on land reclaimed from the river during the late sixties (Fig. 1). Settlement of the area over a period of ten years is expected to have a significant affect on the performance of the pavement, so that the pavement life was chosen as ten years rather than the more normal period of 20 years. This meant that a thinner than normal pavement was designed.

It is expected that the performance characteristics of such a pavement would become evident fairly rapidly. Therefore, it is planned to study this pavement closely as a check on the design method used. The first part of the study was to see how closely the elastic theory was able to predict measurable parameters such as stress and deflection arising from a single wheel load. This paper reports the results of this study although it is not possible here to include a full discussion of all aspects.

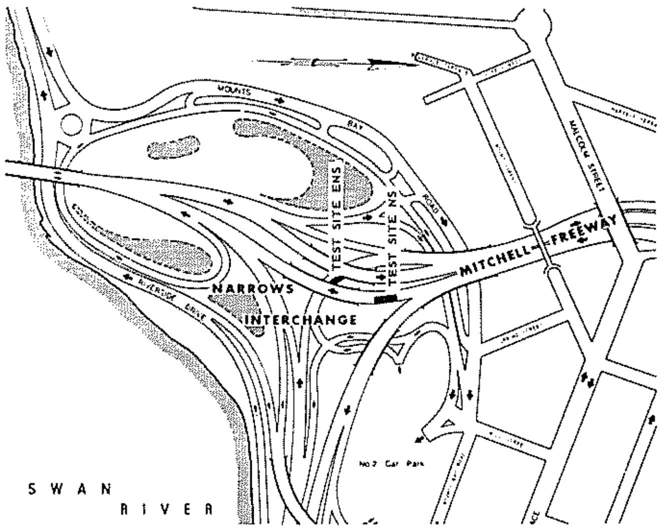

Fig. 1 Location of test sites

2 SITE CONDITIONS

(a) Pavements

Two test sites within the interchange were set up, one (NS) representative of the pavement as a whole, and the other (ENS) representing the singular condition of an unusually high water table. At site N.S. the depth of the water table is in excess of 10 metres. At site ENS the winter level occurs within the subgrade, in summer when all the field testing was performed it is about 500 mm lower. The effect of this variation in water table level on the design pavement thickness at the different sites can be seen from Figure 2. The design thicknesses were 280 mm and 500 mm respectively, and the materials comprising the subgrade and pavements at each site were similar. Both test sites were completed in June 1973.

(b) Materials

The subgrade is a uniformly graded imported quartz sand (C_u = 1.8, C_c = 1.0) with 85% of the particles lying in the range 0.60 mm to 0.15 mm. Subgrades were compacted to a density index of greater than 0.90 for a depth of not less than 300mm. The sub-base is a crushed limestone gravel. This material is very soft and friable and is in fact a weakly cemented calcareous aeolinite, consisting of shell fragments and quartz sand. Calcium carbonate contents vary within wide limits. The material is gap-graded between sizes 19 and 0.0075 mm, and has a deficiency of particles in the range 0.6 - 19mm. It was compacted to densities generally in excess of 95% of the laboratory maximum. The limestone gravel was stabilised with bitumen to allow it to be used as a base course material. Bitumen was added as a 60/40 emulsion with the object of distributing a residual bitumen content of 3% throughout the material. Field densities were generally in excess of 95% of the laboratory maximum. A dense graded bituminous concrete was used as the surface course.

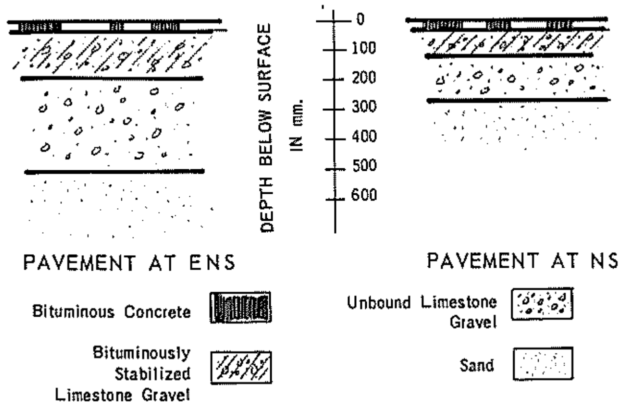

Fig. 2 Test pavement cross-sections

3 FIELD INVESTIGATIONS

(a) Vibratory Testing

The dynamic elastic moduli of pavement components were determined by wave propagation measurements (Ref. 3). Vibrations were induced in the pavements by a small electro-magnetic vibrator and detected by a transducer which is successively located at varying distances from the source. This enables the wavelength of the vibration to be determined and thus the phase velocity. By carrying out tests over a range of frequencies it is possible to obtain the shear wave velocity for each pavement layer and the subgrade. The dynamic modulus is determined from

$$E = 2\rho \ (1 + \nu) \ \beta^2$$

where E = dynamic elastic modulus (kPa)
ρ = density of material (t/m^3)
ν = Poisson's ratio
β = shear wave velocity (m/s)

The tests were conducted on each side of the sites selected for the placement of pressure cells. The observations were consistent and the interpretation straightforward. The results are shown in Table I.

(b) Pressure Cell Measurements

Although it is desirable to install embedded instrumentation at the time of pavement construction, for the particular materials and construction methods involved at these sites, such an approach was impractical. The top of the subgrade was disturbed by placing and compacting the subbase which would give unknown orientation and location of pressure cells, however careful the original installation. In the subbase itself, the wide grading of the material required careful placing of the material surrounding the cell. Therefore it was decided to install all cells after completion of construction

Holes for the installation were drilled using a conventional drill rig and a 200 mm air-circulation

TABLE I

DYNAMIC MODULUS FROM VIBRATORY TESTING

Site	Layer	Velocity (m/s)	Density (t/m^3)	Poissons Ratio	Modulus (MPa)
ENS	Surface	720	2.48	0.4	3600
(OWP)	Base	400	1.95	0.35	840
	Subbase	315	1.95	0.3	500
	Subgrade	205	1.85	0.4	220
ENS	Surface	830	2.48	0.4	4780
(IWP)	Base	440	1.95	0.35	1020
	Subbase	310	1.95	0.3	490
	Subgrade	250	1.85	0.4	320
NS	Surface	700	2.48	0.4	3400
(OWP)	Base	352	1.95	0.35	650
	Subbase	315	1.95	0.3	500
	Subgrade	218	1.85	0.4	250

core cutter specially developed and operated by the MRD drilling section. This recovered a 185 mm core of the wearing and base courses, but because of the low cohesion of the subbase only a few cores of this material were obtained. The pressure cells were placed at the required locations and surrounded by fines of the pavement material before recompacting the bulk material to the estimated in-situ density and moisture content of the adjoining pavement. The leads were taken to a roadside junction box through nylon tubes built into the base of the surface course at the time of construction. The holes were completed with hot-mix.

The pressure cells installed were designed for laboratory use although they have been used in other field experiments. They are 25 mm dia. by 5 mm thick and have a single active diaphragm incorporating conventional resistance strain gauges (Ref. 4). Cells were placed in eight holes of the test sections in Dec. 1973. The performance of these rapidly deteriorated, apparently due to inadequate shielding and waterproofing. They were replaced in March 1974 and the main test series carried out. The pavement loading in these tests was by the dual rear wheels of a truck. The total wheel load was 4050 kg carried on 9.00 x 20 Michelin tyres inflated to 620 kPa.

(c) Deflection Measurements

A Benkelman beam was used to measure the surface deflection of the pavement between the rear wheels of the loading truck. These tests were carried out at the same time as the stress measurements. At site ENS the average rebound deflection was 0.23 mm and at site NS it was 0.43 mm. Load tests were performed using a rigid plate 300 mm in diameter. Deflection was measured by four dial gauges located around the circumference and the load applied by jacking against a box section clamped to the chassis of the loading truck. This limited the maximum load applied to 7000 kg.

A number of load-unload cycles were applied, each time increasing the load until the maximum was reached. At site ENS the response was substantially linear except for the highest loading, the linear response corresponding to a plate deflection of 0.41 mm at the standard load of 4050 kg. The results at site NS showed a large amount of hysteresis at low loads, but the subsequent response was linear and gave a deflection of 0.34 mm for the standard load. In the time available for testing it was not possible to resolve the apparent discrepancy of the ENS result.

(d) Field C.B.R. Tests

Standard field CBR tests were carried out on top of the sand subgrade exposed in six of the holes drilled for pressure cell installation. Surcharge weights of either 13 kg or 18 kg were used and the mean CBR values obtained were for site ENS 32% and for site NS 38%.

4 LABORATORY TESTING

(a) Repeated Load Tests

Modulus values for the granular subgrade and subbase were found from repeated load triaxial tests. The apparatus tests a sample 100 mm diameter by 200 mm high mounted between 'frictionless' ends. Air-operated 'Bellofram' pistons are used to apply both vertical and horizontal stress pulses. The details of the system used to keep these in phase has been previously described (Ref. 5). Transducers inside the cell were used to measure vertical and lateral deflections while external transducers measured cell pressure and vertical load. The modulus determined from the recoverable strain after each stress pulse is usually called the 'resilient' modulus, in this paper it is convenient to use 'dynamic' modulus to describe field and laboratory values used in the subsequent analysis.

Two types of loading were used - one in which the confining pressure was not varied, the other in which it changed in a fixed ratio to the vertical stress. The second better represents stresses applied to a pavement by a wheel load whilst the first has been used by many investigators. The results showed that at the stress levels used, stress history effects were negligible. This is not unexpected with granular materials at the low degrees of saturation used and meant that a single sample could be used to obtain moduli over a range of stress conditions.

The material in the field is loaded by an initial stress arising from pavement weight and any "built-in" stress before it is subjected to the stress increments caused by the wheel load. As a best estimate, the initial stress was taken as isotropic and equal to the overburden stress at the particular level. This was simulated in the laboratory by applying a constant lateral pressure (σ_3) to the sample. The load pulse was then applied on top of this stress, the load cycle being 0.8 seconds on, 1.6 seconds off. A range of initial stresses and applied stress ratios was investigated, based partly on stresses calculated in the elastic analysis.

(b) Subgrade Material

Samples of the subgrade sand were prepared in a three-part split mould in 25 mm layers. The mould was clamped to a vibratory table and the target density obtained by varying the surcharge and time of vibration. To simulate field conditions a moisture content of 8% was used and dry densities over the range 1.63 to 1.72 t/m^3 (dry density index I_D = 0.49 to 0.75) were selected. The highest value corresponds to field conditions and these results were used in the analysis.

The results of tests carried out with constant values of lateral pressure between 7 and 60 kPa are shown in Fig. 3. (Samples with I_D = 0.77 and 0.66). In these tests the maximum vertical stress that can be repeatedly applied is limited by the strength of the sample. When the stress ratio exceeded four,

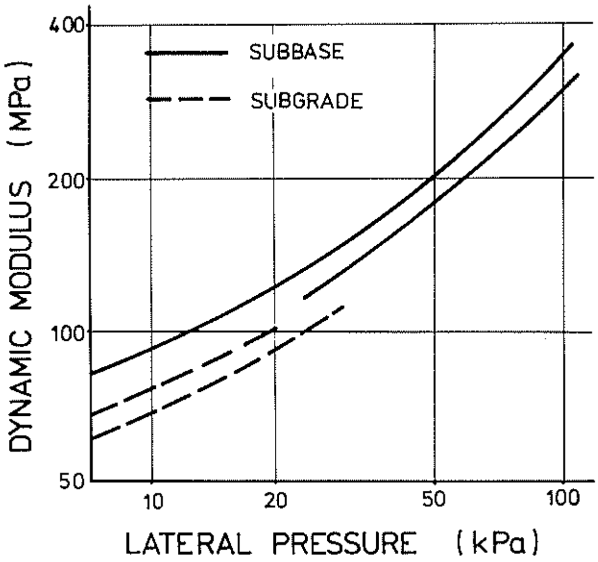

Fig. 3 Dynamic modulus, constant σ_3 tests

significant plastic deformation started to occur under each load cycle while failure occurred at at stress ratio of about five (ϕ' = 42^o). Below a stress ratio of four the strains were recoverable and the dynamic modulus was practically independent of the vertical stress magnitude.

Tests were also performed where the lateral pressure increment was applied in phase with the vertical load. Constant ratios of stress increment (vertical stress increment ÷ horizontal stress increment) of 6, 4 and 3 were used. The actual stress ratios are different since an initial stress also acted. Test results for the sample at field density are shown in Fig. 4. The dynamic modulus now depends on initial stress and vertical stress increment but is practically independent of the stress increment ratio over the range considered. The results at other densities showed similar trends but the actual modulus was lower.

Values of Poisson's ratio assuming isotropic behaviour were also calculated from these tests. Direct measurement of radial deflection proved unreliable but the sample volume change could be obtained. (This was an air volume change because of the low degree of saturation). The results gave ν = 0.5 at low levels of vertical stress increment but the value decreased with increasing stress increment and stress increment ratio, and decreasing lateral pressure. The value was zero for some of the stress combinations in Fig. 4. At lower densities the pattern was similar but the minimum value was about 0.1.

(c) Subbase Material

The dynamic modulus of the limestone subbase was obtained using the repeated load apparatus. The target densities were achieved by a combination of dynamic and static compaction, placing the material in 12 mm layers in the mould. Moisture contents between 9% and 10% were used, but MRD experience has been that stiffness is very little influenced by moisture content. The results of tests at constant lateral pressure are shown in Fig. 3 (Samples at ρ_D = 1.78 and 1.70t/m^3, 2 hours old) and at various constant stress increment ratios in Fig. 5. (ρ_D = 1.78t/m^3, 5 days old). These show similar trends to those observed with the sand.

Two cores of the subbase were available, but because of their large diameter could only be tested in unconfined compression with slow load cycling. Both samples showed markedly non-linear behaviour, larger relative strains occurring at low stress levels. This effect was not due to bedding errors at the plaster capping, since 'DEMEC' and LVDT strain measurement directly onto the sample surface showed identical behaviour. Over the range of vertical stress increments reported in Fig. 5, the mean secant modulus varied from 290 to 680 MPa.

It was apparent that time-dependent effects were present, presumably associated with the natural cementitious properties of the material. Therefore, one sample was tested in the repeated load apparatus at times of 2 hours, 1, 5 and 20 days after compaction. At each age the modulus showed the same pattern as in Fig. 5, but the values consistently increased with age. From all these results the modulus appropriate to the pavement age at the time of the field tests could be estimated.

Main Roads Department laboratory records for Texas Triaxial tests on similar limestones at these densities were also examined. These showed compression modulus values of about 30 MPa unconfined rising to about 70MPa at a confining pressure of 70 kPa. These values are significantly lower than those measured in the laboratory repeated load tests.

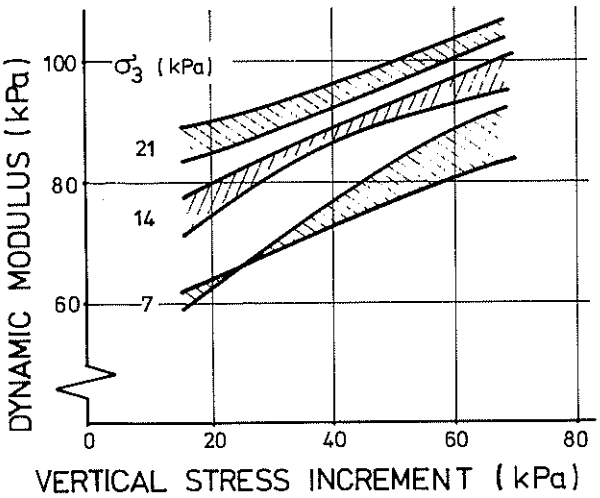

Fig 4 subgrade modulus, constant σ_1/σ_3 tests

5 ANALYSIS

(a) Modulus Values

From the results in Figs. 3,4 and 5, modulus values can be selected appropriate to the stress conditions and material densities of the subgrade and subbase. For the latter an interpolation based on age is also necessary. Since initial stress and stress increment varies with depth throughout the pavement the modulus will vary also. Because a linear elastic analysis is proposed, a mean value is taken.

The results from the field and laboratory tests for the subbase and subgrade are shown in Table II. The CBR of the subgrade can be used to estimate a dynamic modulus using the relationship

$$E(\text{kgf/cm}^2) = 100 \text{ CBR} \quad (\text{ref. 6}).$$

The two types of laboratory repeated load testing give similar values thus justifying the use of the simpler approach if linear elastic analysis is proposed. Comparing field and laboratory values there is good agreement for the subbase but very large differences for the subgrade. From Figs. 3 and 4 there appear to be no feasible stresses which could lift the laboratory values sufficiently to agree with the field. A more likely interpretation of the field values is that they represent the properties of an intermediate layer comprising subbase contaminated with the subgrade sand which has been reused from elsewhere on site as part of the stage construction process. Measurements in the cored holes indicated that subbase material extended to a depth of 380 mm at site NS and to 720 mm at ENS. Part of the depth in excess of design is probably part of the normal construction tolerance, but for analysis, 100 mm of the excess is taken as the thickness of the intermediate layer.

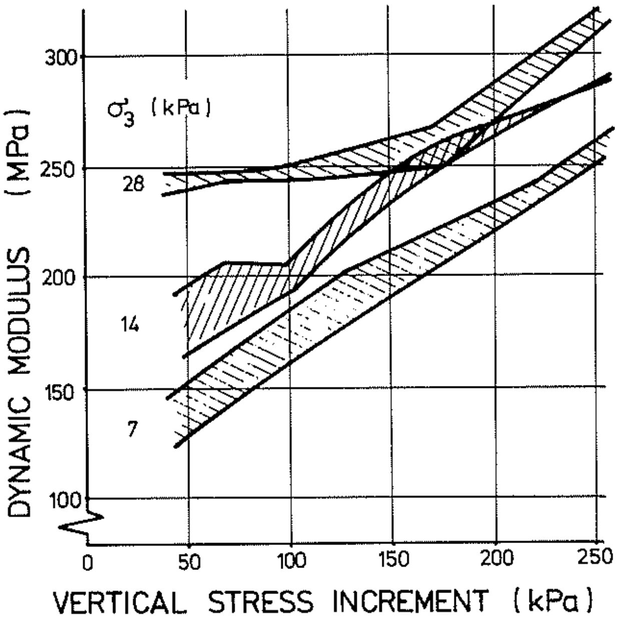

Fig. 5 Subbase modulus, constant σ_1/σ_3 tests

(b) Elastic Analysis

One of the aims of the test program was to compare predictions of stresses in and deflections of pavements with those measured. Many theoretical models have been proposed, here only that in which the pavement is assumed to comprise linearly elastic homogeneous layers will be considered. (These assumptions are made in the Shell design method). The idealised sections analysed are similar to those shown in Fig. 2 with the modifications discussed above. Actual layer thicknesses as measured in the cored holes were used. The laboratory values have been taken from Table II, where these are not available, Table I values were used. Analyses were made using program 'CRALAY' (ref. 7) which gives results identical with the Shell program, is readily accessible and quite versatile.

TABLE II

VALUES OF DYNAMIC MODULUS (MPa)

Layer	Laboratory Testing Constant σ_3	Laboratory Testing Constant σ_1/σ_3	Vibratory Testing	Field CBR
Subgrade	90	80	260	360
Subbase	400	450	500	-

The predicted vertical stress distribution in pavement ENS is shown in Fig. 6. The higher stress curve near the surface represents conditions beneath one tyre of the dual wheel, the other curve is for stresses midway between the tyres. Below 200 mm the difference is negligible. Over the depth for which field measurements are available the vertical stress distribution in pavement NS is almost identical, so is not shown.

The measurements made in each pavement are also shown in Fig. 6. The bar indicates the range of values mainly arising from differences in lateral location of the dual wheels as they passed over the pressure cell location, but also including other errors. The predicted and measured stresses are in reasonable agreement down to about 300 mm, below which the measured values are less. The semi-logarithmic plot accentuates the differences there, which are in absolute terms not very large. The values may be influenced by the lower modulus of the hole backfill but studies of the lateral dispersion of the vertical stress indicated that this effect was not large. Overall, the agreement between predicted and measured values is reasonable.

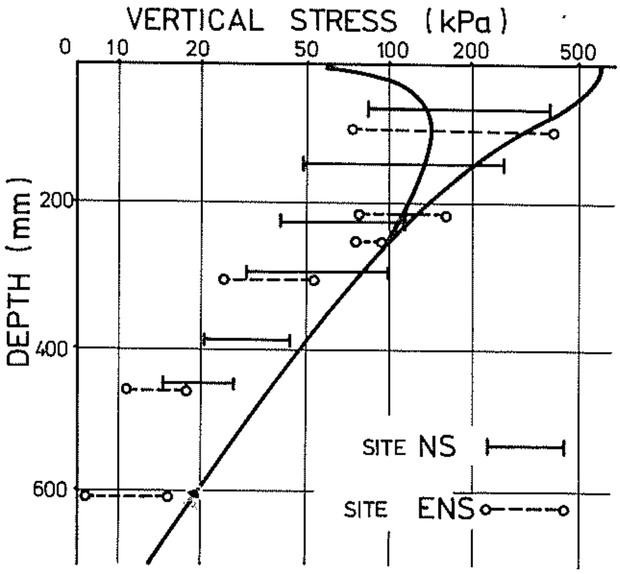

Fig. 6 Vertical stress distributions

Whereas the stress distribution in the pavement is largely controlled by the base and subbase moduli, deflections depend much more on the subgrade value. Using the idealised sections, surface deflections were calculated between the tyres for comparison with the Benkelman beam observations. The results were 0.22 mm (0.23) for section ENS and 0.32 mm (0.43) for NS, the figures in brackets being measured values. An estimated plate load deflection was also determined by correcting the flexible load solution produced by CRALAY for the effect of a rigid plate. The results were 0.20 mm (0.41) for ENS, and 0.33 mm (0.34) for NS. There are obvious discrepancies in both groups of measured values as can be seen by comparing the relative values for each section measured in each test and also for the particular test results on each section. Overall the predicted values appear to be of the right order, any significant changes to the subgrade modulus gives substantially worse agreement.

6 CONCLUSIONS

The discussion has indicated some of the difficulties associated with full-scale tests on in-service pavements. Cost and access problems mean that tests must be limited in number and the results are therefore subject to more error than may be desirable. However, without such tests the science of road design will not advance and these tests have made a positive contribution to that end.

Laboratory repeated load testing of the subbase and subgrade granular materials gave modulus values which appeared appropriate for use in the subsequent elastic analysis. These tests also indicated how the modulus varied with stress (Figs. 3,4,5) and showed that for linear elastic analysis results from tests at constant lateral pressure are adequate.

The field vibratory testing gave good estimates of the moduli of the pavement layers but it appears that the modulus of the true subgrade was not detected. This problem should not occur with a cohesive subgrade where the boundary is clearly defined, whereas here an intermediate layer was present.

Elastic analysis of the same type as used to set up the design charts was employed for prediction methods. This was easy to apply using a combination of field and laboratory-determined moduli. Reasonable agreement was found between predicted and measured vertical stresses and surface displacements. Thus, for this type of pavement, elastic analysis can be used, although the tests did not give any indication of the adequacy of the failure criteria employed with the Shell design method.

7 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of the ARRB through Mr. D. Potter who performed the field vibratory tests. Messrs. G. Elderfield and J. Greenwood of the MRD carried out the field instrumentation and tests. The paper is published with the permission of Mr. D. H. Aitken, B.E., F.I.E.Aust, F.A.I.M., F.C.I.T., Commissioner of Main Roads, W.A.

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