THE PERFORMANCE OF UNBOUND BASECOURSE
UNDER SIMULATED TRAFFIC

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SYNOPSIS

The performance of unbound granular basecourse surfaced with chip seal has been studied in a field situation using simulated traffic loads. Test variables included subgrade deflection, aggregate type, grading and shape.

Pavement distress due to basecourse instability may be attributed fundamentally to thickness change due to densification and/or lateral shear movement of material beneath a wheel path. Contrary to reports involving laboratory models, the field trials found that lateral movement provided no measurable contribution to rut depth unless either high saturation or excessive pavement deflection had occurred. The most significant factor affecting the stability of basecourses was densification at constant moisture content resulting in a high degree of saturation (over 80%). This produced progressive rutting and lateral shear. The character of highly saturated basecourses was deceptive: excavated specimens were strongly cohesive and appeared quite "dry". (Testing showed that moisture contents as low as 3.5% give excessive saturation in some basecourses after less than 10% of design traffic loading). The most adequate means of predicting susceptibility of stockpile material to excessive densification is examination of the shape of the particle graduation curve. The sand equivalent test, which reputedly determines the presence of undesirable fine material in aggregates, was found to correlate poorly with basecourse performance. Guidelines are offered for the practitioner wishing to utilise local materials, which do not comply with current NRB requirements for basecourse or subbase.

1. INTRODUCTION

This project was proposed in order to study the performance of unbound granular basecourses using the existing facilities at the University of Canterbury pavement test track. The testing facilities have been described by William and Paterson (1971). The track is circular and may be trafficked using two loaded vehicles attached to the ends of a centrally pivoting 'axle'. The pivot itself moves in 0.6m diameter orbit. This results in a lateral distribution of successive wheelpaths on highways. Each vehicle is loaded to 4.1 tonne and is carried by a dual wheel with tyres inflated to 520kpa. The stresses induced in the pavement are thus comparable to those beneath the wheels of a Class I axle loaded to the legal limit. The machine is powered by an 8kW Demag electric motor, which drives one vehicle the other being free wheeling. Normal trafficking speed is 15km/hr.

2. THE TEST PAVEMENTS

2.1 General

Two pavements were constructed. The first series of tests was on a soft loess subgrade, which provided a highly yielding pavement. The pavement for the second series was constructed on a firm gravel subgrade (see figure 1).
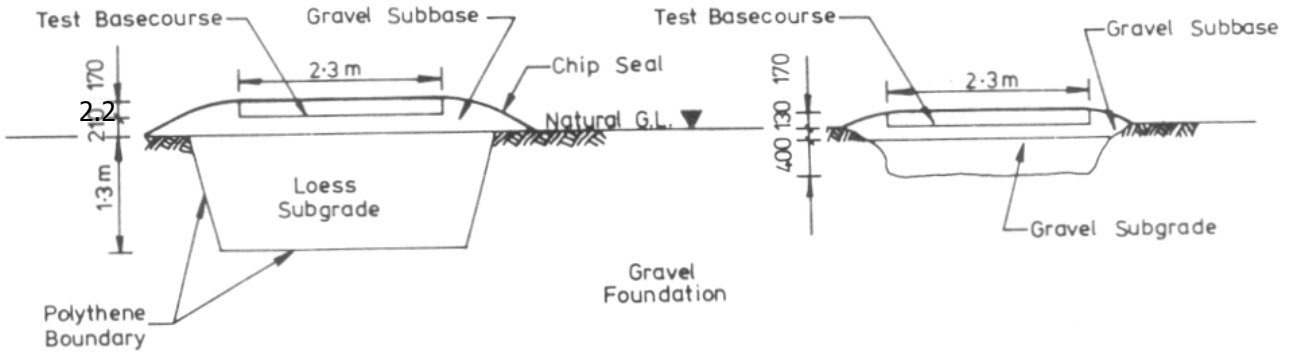

Figure 1 (a)   Series I track.
Yielding pavement
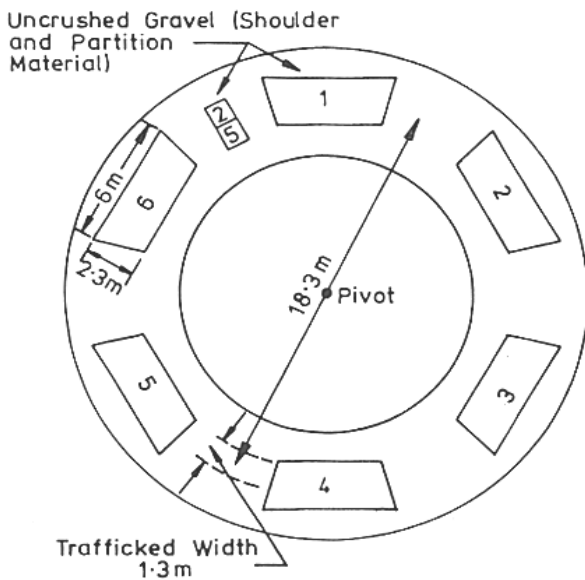
Figure 1 (b)   Series II track.
Normal pavement

Aggregates Tested

1. Q n.65
2. Q n.55
3. Q n.5
4. C95 n.65
5. C94 n.5
6. C94 n.4
7. C71 n.5
8. C59 n.45
9. C50 n.5
10. C50 n.4
10(a) C77 n.55
11. R   n.4
12. R   n.35
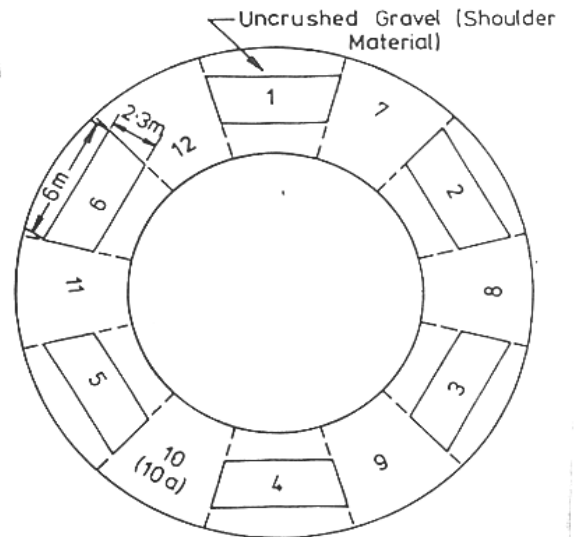
Figure 1 (c)   Series I track.

Figure 1 (d)   Series II track.

<u>Aggregates</u>

In the first series of tests, rock from Stevenson and Son's quarry at Drury was used in three gradings, which followed as close as practicable to the coarse, average and fine regions of the NRB M4 (1975) basecourse specification. Marshall (1974) describes the Drury rock as medium to coarse siltstones with lesser amounts of fine sandstone and argillites. The aggregates supplied were considered comparable with the best obtainable from the quarry.

Crushed gravels to approximately the same gradings as the quarried rock were supplied by British Pavements (Canterbury) Ltd from their Miners Road pit near Christchurch. These stones are predominantly well indurated greywacke sandstones with minor amounts of finer grained siltstones and argillites.

In the second series, an additional range of both rounded and partly crushed gravels was laid as well as further material from the six aggregates used in Series I.

To simplify later discussion the basecourse will be referred to by stating the percentage of broken stone (C) and average grading exponent (n), Talbot and Richart (1923). The exceptions will be quarried rock (Q) and the uncrushed rounded gravel (R). The gradings and corresponding reference codes are contained in the Appendix, n has been found by extrapolating the grading curve drawn on log-log paper to find the effective maximum size (D). The gradient of the best fit line from the co-ordinate (D, 100) through the data, gives n. A listing of all aggregates tested is contained in Figure 1.

2.3 <u>Field Testing</u>

2.3.1 <u>Deflection</u>
Benkleman beam readings were taken using the method described by Belshaw (1971). Deflections were measured at three locations on each basecourse.

2.3.2 <u>Surface Profiles</u>
For Series I, profiles were initially measure using the recorder described by Paterson (1972) to plot a continuous trace of the surface shape across marked sections. Pavement deformations soon became so severe that they could no longer be accommodated in the range of the recorder. In Series II, rutting was measured on all basecourses at the centre of the wheelpath. Measurement was repeated at the same points during the trial with the use of a 2.5m long frame, which could be accurately relocated across the wheelpath. Precise levelling of the datum points, on which the frame was placed, showed that the 2.5m span was sufficiently long to give results unbiased by deep settlement. The rut depths recorded in the Appendix are the averages of nine measurements made at the centreline of each basecourse. Edge heave was similarly measured at points of 50mm and 100mm beyond the edges of the tracked surface.

2.3.3 <u>Density</u>
The balloom densometer method was used for most density testing with check tests using both sand replacement and wax coating of block samples.

2.3.4 <u>Sampling for Laboratory Tests</u>
All samples were extracted from parallel-sided trenches occupying the central third of the1.3m wide trafficked wheelpath. The full depth of basecourse was removed down to a fabric layer positioned at the basecourse/subbase boundary. A minimum of 50kg was taken for subsequent riffling and testing.

3. PAVEMENT PERFORMANCE

3.1 Series I

The data for individual basecourses are recorded in the Appendix.

With such a channelised loading, some densification rutting developed around the whole circuit. Much more obvious was the more substantial rutting and matching edge heave associated with lateral creep. The performances of the test sections were "undermined" by the high deflections, which had developed, by 10,000 EDA.

In particular, the variation in the magnitude of pavement deflection around the track dictated the performance of overlying basecourse. Deflection value at any point controlled the rate of permanent deformation occurring within the portion of whatever basecourse was in that immediate locality. Every measured point which showed an early life deflection exceeding a critical value of 1.50mm suffered a substantial increase in deflection with pronounced wheelpath rutting and edge heave.

To minimise impact suffered by the test machine, depressions were filled during trafficking while testing continued to 66,000 EDA although it was clear that this would mask shape change and would also affect the traffic stresses on the basecourse. Even so, at the end of the test, deformation of the original surface was almost completely deflection correlated. For any point, performance was found to be best characterised by the maximum deflection observed during the trial at that point. It was noted that al all points where the characteristic deflection exceeded 2.0mm there was visible edge heave and rutting. Elsewhere, in the absence of edge heave, rutting was limited to 20mm.

Relevant to the failure mechanism discussed below was the extremely high deflection encountered on a short length of the circuit. At 10,000 EDA, deflection had risen to 10mm. Edge heave occurred steadily and plant mix (a cold compacting fine asphalt) was used to infill the rut, although after progressive compaction it too could not tolerate the excessive strains and moved along the wheelpath. A mixture of all passing 14mm crushed gravel with a small amount of plant mix was then used for filling. The rate of shear then began to decrease and further heave had apparently ceased by the end of the test. Although difficult to measure on the loose surface, final deflection had reduced to about 2.0mm.

The pavement was finally dug out to examine the shape of the sheets of fabric place between each layer, with most interest in observing deformation of the soft subgrade. The upper fabric layer, between basecourse and subbase was found to conform with the surface deformation, showing rut and heave in sympathy. The subbase was noticeably harder and more difficult to excavate at areas of significant rut and heave: in other words, from the previous observations, high deflections had indicated areas where the subbase was hardest! The fabric at the subbase – subgrade interface was then exposed but at no section could any significant continuation of the rut patter be observed.

3.2 Series II

Surface deformations in Series II were mush less dramatic than in the earlier trial, with one notable exception. The C50 n.4 showed no initial instability in the first few 100 EDA, but a minor increase in deflection was noted at 1,000 EDA. By 3,000 EDA rutting and lateral heave were apparent. Soon each wheel-pass left distinct impressions but the section was infilled with plant mix until testing was next planned at 16,300 EDA. When excavated, the basecourse showed considerable cohesion and was surprisingly hard.

 A new test aggregate C 77 n.55 was used for a replacement. This was compacted and sealed in a similar manner to the remainder of the track.

Trafficking was continued for 312,000 EDA. Deflection decreased to a very uniform 0.5mm on all sections. Rutting progressed to a similar depth of about 10mm in each aggregate with the exception of the replacement aggregate, which rutted only 5mm. At the end of trafficking there was very little visual difference around the track.

4. DISCUSSION

4.1 General

Pavement distress and its manifestations have been discussed by several authors and are reviewed in RRU Bulletin No.21. The basic objective in construction is to form a pavement using materials and methods so that minimal change occurs in the thickness of any layer during trafficking. Provided the seal coat remains, thickness change may occur only by (a) change of density, which is usually compressional but may be dilational or (b) lateral movement of material away from the wheelpath. The latter mechanism is reported to result from accumulation of residual shear strains during repeated flexure.

Since densification will be discussed at considerable length, the concept of voids will be used where quantitative results are reviewed or presented. The percentage of voids is the volume of air and water contained between particles within an aggregate, expressed as a percentage of the total volume. Saturated surface dry density of solids has been used in calculations. It has been assumed that absorbed water has not contributed to the degree of saturation of voids between particles.

4.2 Distress Mechanism

4.2.1 Series I.
The extreme surface distress produced in Series I resulted from progressive lateral shear in the subbase. Densification at constant moisture continued within the pavement until the subbase reached a critical saturation level with increased pore water pressures and loss of shear strength under load. The deformation described in 3.1 indicates that instability originated in the subbase since this was the only component showing significant change in thickness. Although directly tested, high saturation conditions have been inferred from the performance discussed below.

4.2.2 Series II.
Distress in Series II was limited to the early failure of C 50 n.4 and deformation of C 59 n.45. When C 50 n.4 was excavated, the unaffected fabric layer above the subbase confirmed that the excessive surface deformation was caused solely by loss of strength within the basecourse. The aggregate showed the same hard, highly cohesive characteristics that had been found in the Series I subbase.

An intact sample from C 50 n.4 after 16,300 EDA was used to provide an accurate determination for the degree of saturation by using a wax coating for calculation of volume. The sample contained 7.8% total voids and was 90% saturated. Instability under these conditions is consistent with the finding of Haynes and Yoder (1964) and Barenberg (1971) who found that cyclic shear with saturation in excess of about 80% caused large increases in the rate of creep deformation.

The localised rutting in C 59 n.45 was also due to high saturation. That this aggregate did not suffer continued deformation might be attributed to the interruption of trafficking in September, when field tests were made. The next tests, done in December, showed significant reduction in moisture content with consequent reduction in saturation.

4.3 Degradation

The soundness of the aggregate resulted in no consistent gradation change in the pavement sections, however, sand equivalents dropped markedly. At the end of Series II, after 300,000 EDA, all but one of the aggregates gave sand equivalents of less than 30. Most values were close to 25.

4.4 Densification

Each basecourse approached a refusal density or refusal voids after sufficient EDA. The actual value and number of EDA to achieve refusal are both clearly dependent on the strain characteristics of the pavement system: voids achieved in Series I at 16,000 EDA are all less than or equal to refusal voids of Series II. This is a result of the higher shear strains caused by the more yielding subbase and subgrade. Similar trends have been found by Youd (1972) and Pyke (1975) who examined the densification of sands under cyclic shear.

Densification, resulting in a high degree of saturation, is the most significant factor affecting the resistance to thickness changes of an unbound basecourse. Excessive fines, measured by percentage passing 0.075mm and sand equivalent, may hold sufficient levels in a basecourse after densification caused by traffic compaction. It appears that a reasonably durable basecourse will reach a refusal density at a rate depending on the strain characteristics of the pavement, however for typical pavement configurations refusal density may be expected to be reached at less than 10% of design life. The densification to saturation problem has been noted by Buckland (1965) and Bartley (1975).

4.5 Shear Resistance

Shear resistance is here discussed for drained conditions – that is, in the absence of pore water pressures. Lateral heave in Series I appeared to be related to densification – saturation instability in the subbase.

It has already been noted that at the end of Series I, no lateral heave was observed wherever deflections had not exceeded 2.0mm. It is not clear why at 66,000 EDA some sections were stable while any section having deflection over 1.5mm at the start of the test had shortly developed a subbase failure. Non-uniformity in thickness or fines content of the subbase is a possibility but appears unlikely in view of the construction method. Localised but deep-seated areas of softening of the subgrade due to ingress of rainwater appears a more probable reason. This would account for the increased vertical deflection in the absence of excessive shear strains, which would have densified the subbase. This could have been verified, had deflection bowl shapes been recorded throughout the trial.

To obtain more precise information on lateral shear from edge heave, reference points to measure heave on Series II were set out as described in 2.3.2. To the accuracy of measurement ($\pm$ 1mm) no movement was detectable, with the notable exception of the densified C 50 n.4.

The demise of C 50 n.4 and its replacement with C 77 n.5 provided further information on the lateral creep. The Appendix shows the actual amount of rutting calculated from and attributed to the basecourse densification as well as the total measured rut. Since C 77 n.55 was introduced at 16,300 EDA, traffic densification of the lower layers had already been largely effected. It may be seen that the 5mm of rutting in C 77 n.55 after 312,000 EDA is almost entirely accounted for by basecourse densification: only 1.4mm has occurred due to other causes (deeper densification and lateral creep).

In this project, there has been no observation of lateral creep except where high saturation or excessive deflection occurred.

The writer considers that much published data from laboratory testing of basecourse materials under cyclic loading is unrelated to field performance of unbound granular basecourses under sensible pavement configurations. (Sensible implying no extremes of deflection; say, deflection less than 1.5mm and no abnormally high curvature of the deflection bowl). It is clear that the early life densification of a basecourse under the stress reversals occurring in a real pavement have not been appreciated. Cases are discussed in Salt (1977). Thompson (1969) and Barenberg (1971) found that permanent strain in cyclicly loaded aggregates is much more dependent on dry density than other parameters. Toan (1975) also notes that a 3% reduction in density causes a drop of about 25% in resilient modulus. His samples of basalt were compacted to a standard density of 2.35t/m. The density of solids was recorded as 3.05. Assuming a typical water absorption for Auckland basalts of 2.5% effective voids of 17.1% are inferred. This is a realistic voids content for an as-constructed field sample. After some 10,000 EDA, voids will be about 11 to 12% for the grading concerned. The continued rate of rutting for the remaining life will relate to this value. Toan gives some cycles of simulated traffic loading, the change of density was very small (0.3%) and this was dilational!

4.6 Effect of Fines

The role of fines, measured by percentage passing 0.075mm and sand equivalent, is seen primarily as providing a large surface area for attraction of moisture, hence dictating the equilibrium moisture content.

The lack of knowledge regarding what sand equivalent really measures is providing difficulties with acceptance criteria for Canterbury aggregates, which meet NRB M4 criteria in all other respects.

The test appears to be written into the M4 specification to ensure that an aggregate will be non-plastic. This has led to the suggestion that if an aggregate fails the sand equivalent criterion, it may still give acceptable performance if it is non-plastic.

The amount of fine material acceptable in a basecourse is determined by its refusal density. Comparison of all gradings tested suggests the reason for the denser gradings is that their proportion of coarse and medium sand is relatively low compared to the amount of finer material. Such distributions produce similar grading shapes to the maximum density gradings found by Fuller and Thompson (1907) and shown in Figure 2. Consequently the Talbot and Richart exponential gradings which have been adopted as the basis for the NRB M4 grading limits require too much sand to allow maximum density. Stresses imposed on compacted aggregate with exponential grading will be transmitted through the sand grains rather than the finer material. Use of some grading shape control will always be of advantage in limiting the gradation change. The remaining requirement to ensure stability is that the very fine sand and silt particles and the water associated with their relatively large surface areas, do not take up so much space between larger grains that a high degree of saturation can result.

Sufficient sand in basecourse is currently being ensured by the M4 grading shape controls and the sand equivalent. A more rational approach could result from the study of pavements with good drainage yet showing shallow shear deformation and basecourse saturation. From the limited data available the ratios of percentages passing the following sieves appear to be significant:
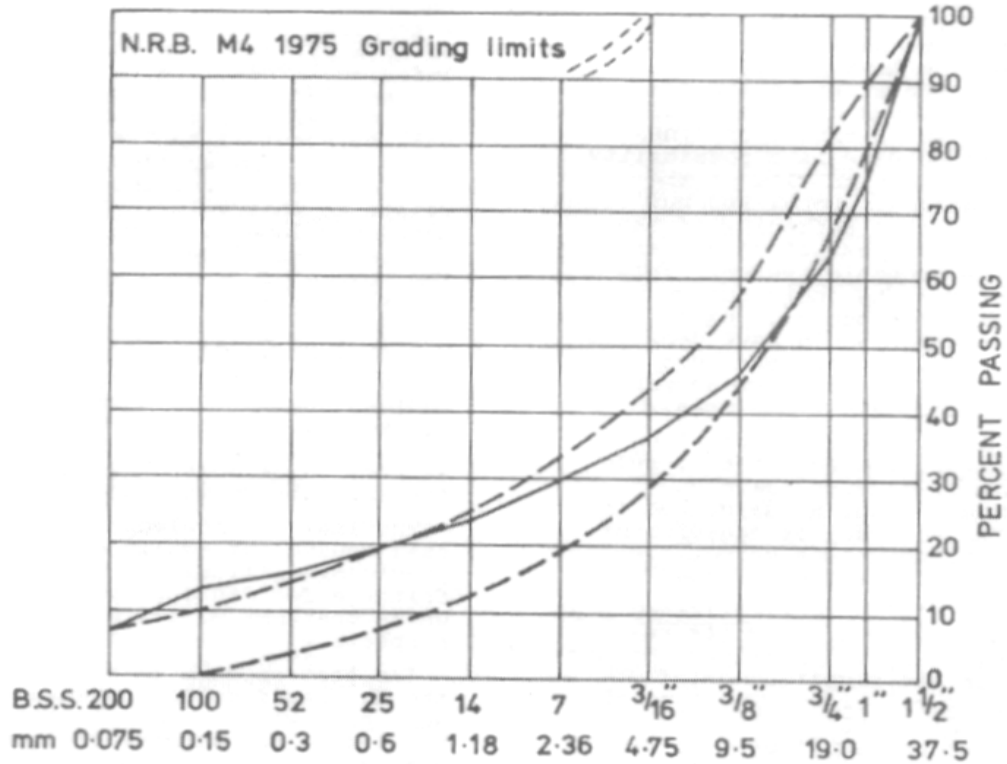

Figure 2. Maximum Density Grading. After Fuller and Thompson.

$R1 = (P2.36mm - PO.15mm) / PO.15mm$
 and
$R2 = (P4.75mm - PO.3mm) / PO.3mm$

(R1 alone is significant in the majority of cases; however, a combination of R1 and R2 provides more complete evaluation).

The following table shows the relation between these ratios and saturation category for Series II.

Aggregate	R1	R2	Saturation	Comments
Q n.55	5.8	5.0		Aggregates with less than 75%
Q n.65	3.5	4.1		saturation showed no instability
C 71 n.5	2.7	2.2	LESS THAN 75%	in 312,000 EDA.
C 50 n.5	2.7	1.2		
C 94 n.4	2.6	2.6		
Q n.5	2.5	2.9		
C 94 n.5	2.5	3.0		
C 77 n.55	2.1	1.3		
C 95 n.65	1.7	2.2		- - - See Below
R n.4	2.3	0.85	75–80%	
R n.35	1.86	0.7		
				- - - Slight deformation at
C 59 n.45	1.84	1.2	OVER 85%	16,300 EDA
C 50 n.4	1.0	0.65		- - - Excessive deformation at
				3,000 EDA

The R1 and R2 ratios both indicate the relative amounts of "load-carrying" sand grains, to the void filling, moisture holding fines. The ratios are hence predictors of the likelihood of a trafficked basecourse achieving a high degree of saturation and consequently exhibiting instability. A basecourse with low sand to fines ratios (R1, R2) can densify to a state in which there is insufficient space between the load-carrying grains to freely accommodate the finer material with its associated moisture.

The existing M4 grading limits and shape control require that $R1 \geq 0.9$ and $R2 \geq 0.86$. The R2 limit of 0.86 appears satisfactory (though marginal) for the above data. The R1 limit of 0.9 is clearly too low to safeguard against a densification – saturation condition.

The criteria for sand equivalent and plasticity have tended to reduce the possibility of accepting basecourses that will readily become saturated but a more positive determination is suggested. From BC-15 experience the following limits are proposed:
R1 < 1.8 or R2 < 0.8 - - low performance expectation
R1 > 2.0 and R2 > 2.0 - - high performance expectation
Allowing engineering assessment between these limits by giving consideration to (i) design EDA, (ii) aggregate durability and (iii) the strain characteristics of the pavement configuration.

The only apparent inconsistency in the above table is C 95 n.65. Saturation less that 75% is noted yet R1 = 1.7 which is unacceptable by the proposed limits. The grading curve for this aggregate shows the excessive amount of coarse (plus 4.75mm) material in the grading. Internal stresses are transmitted only via the coarse material giving a basecourse with excessive 'point' loading hence encouraging degradation. The aggregate is thus appropriately rejected on the R1 value and existing M4 grading limits.

It is of interest to note that the two basecourse failures on the Auckland Motorway, discussed by Buckland (1965) and Cornwell (1966) are consistent with BC-15 findings. Buckland attributed the failure of the Redoubt Road section of the motorway to densification-saturation. Cornwell attributed failure of the Takanini section to degradation and densification but without saturation. The final gradings recorded by Buckland for these sections show:

Redoubt section, R1 = 1.6 (<1.8 hence saturation likely)
Takanini section, R1 = 2.1 (>2.0 hence saturation unlikely)
(R2 satisfactory for both)

For the subbase which became saturated and failed in Series I of the project R1 = 1.4 and R2 = 0.75 (saturation likely).

The need for grading shape criteria (such as R1 and R2, which may be somewhat dependent on petrology and climatic factors) is evident from data recorded in this project. A compacted trafficked basecourse will contain material that yields a sand equivalent value, which has little correlation with the stockpile test. The variety of basecourses used in Series II had stockpile sand equivalents with a mean of 38 and a standard deviation of 17 (see Appendix). At 300,000 EDA (a small portion of design life) the mean was 26 and standard deviation 9. It is clear that basecourses with initially high sand equivalents show rapid decreases (test values almost always below 30) after minor trafficking; while those initially showing low values, change very little.

The role of "Plastic" fines is regarded by the author as primarily the holding of moisture (by surface attraction) hence encouraging saturation and consequent pore pressures under dynamic loading. Purely mechanical "lubrication" has insignificant effect in densely interlocked typical basecourse. Plastic fines (whether above or below their plastic limit) will not detrimentally affect a pavement unless a high degree of saturation exists through a deficit of sand (i.e. R1, R2 not satisfied). It may be noted that these two parameters are relatively insensitive to gradation changes from segregation during placement: field operation may produce considerable shifts in the grading curve position, but will have much less effect on the grading shape of the fined (passing 4.75mm) fraction used for computing R1 and R2.

4.7 Loadspreading Ability

Loadspreading ability has been regarded as a fundamental desirable property of basecourses and subbases in order to provide minimal strains in subgrades. Differences in loadspreading ability between basecourses of equal depth on the test pavement should be reflected by their respective defections under load (assuming the majority of the defection occurs beneath the basecourse layer).

Series I began with excellent uniformity although deflection was high (1.25mm), but deflections after trafficking ranged widely. The Series II pavement had an early life deflection of 0.8mm with fair uniformity but settled with trafficking until a deflection of 0.5mm was obtained, with excellent uniformity.

Deflections at various stages within the trials were compared with density, grading and proportion of broken stone, but no distinction between basecourses could be detected. Results from this project suggest that loadspread in unbound basecourse is a function of aggregate thickness only.

It appears that the cost of crushing gravels to produce 70% of broken stone cannot be justified by the resulting effect on loadspreading.

5. CONCLUSIONS

The following summary of earlier discussion gives the conclusions applicable of aggregates and varied testing conditions should enable some general application to typical unbound granular pavements.

5.1

Densification resulting in a high degree of saturation is the most significant factor affecting the resistance to thickness changes of an unbound basecourse. Excessive fines may hold sufficient moisture to provide critical saturation levels in a basecourse, after densification caused by traffic compaction. It appears that a reasonably durable basecourse will reach a refusal density at a rate depending on the strain characteristics of the pavement. However, for typical pavements, refusal density will be reached at less than 10% of design life.

The practitioner wishing to construct a subbase or basecourse utilising readily available materials which deviate only slightly from NRB requirements for durability, grading or sand equivalent, may find an acceptable local variant, provided he safeguards against the commonly seen shallow shear type failure by using only those materials with adequate grading shape. (i.e. R1, R2 values satisfied).

5.2

Lateral creep of unbound granular material, subject to repetitive traffic loading, provides no measurable contribution to rut depth except where high saturation or excessive deflection of the pavement had occurred.

5.3

A compacted trafficked basecourse will contain material that yields a sand equivalent value, which has little correlation with the stockpile, test. More realistic acceptance criteria for limitation of fines in basecourse can be obtained from parameters, which provide additional grading shape control.

5.4

In crushed gravel basecourses, actual percentage of broken stone has little effect on performance. With less than 50% of broken stone, some increase in susceptibility to excessive densification occurs but this can be offset by more restrictive grading limits. Beyond a practical minimum of 50%, no superiority is shown by basecourses containing a greater percentage of crushed material.

5.5

The results of any laboratory investigations of compacted aggregates cannot be meaningfully applied to real pavements unless densities similar to trafficked materials have been achieved.

6. ACKNOWLEDGEMENTS

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Appendix

PERFORMANCE DATA

AGGREGATE	Q	Q	Q	C95	C94	C94
	n.65	n.55	n.5	n.65	n.5	n.4
SERIES I : 66,000 EDA						
As compacted voids (%)	15.7	18.0	14.7	15.0	16.0	14.3
Refusal Voids (%)	10.5	10.0	9.5	6.5	8.0	8.0
SERIES II : 300,000 EDA						
As compacted voids (%)	18.0	15.4	15.2	15.6	14.0	13.3
Refusal voids (%)	12.8	12.7	10.0	10.0	10.6	9.3
Stockpile sand equivalent	66	58	26	53	34	33
Terminal sand equivalent	53	29	24	27	25	27
Total rut depth (mm)	2.5	2.0	4.5	6.2	4.1	3.2
Rut due to basecourse densification (mm)	2.5	2.0	4.5	6.2	4.1	3.2
Rut not due to basecourse densification (mm)	7.5	9.0	5.5	4.8	7.9	4.3

AGGREGATE	C77	C71	C59	C50	C50	R
	n.55	n.5	n.45	n.5	n.4	n.4
SERIES II : 300,000 EDA						
As compacted voids (%)	12.0	12.5	11.0	13.4	11.0	11.0
Refusal voids (%)	9.9	8.5	6.0	9.3	7.3	7.8
Stockpile sand equivalent	25	40	19	61	30	16
Terminal sand equivalent	22	23	19	28	22	14
Total rut depth (mm)	5.0	11	10	9	9	**
Rut due to basecourse densification (mm)	3.6	4.3	7.0	6.5	3.5	4.5
Rut not due to basecourse densification (mm)	1.4*	6.7	3.0	2.5	5.5	**

NOTES: 1. * See text on shear resistance
2. ** Early shear failure
3.   Rut depth origin established at 100 EDA