6. G.H. Hilt and D.T. Davidson. Lime Fixation in Clayey Soils. HRB, Highway Research Bull. 262, 1960, pp. 20-32.
7. A.J. Hanks. Frost Heave Measurements of Lime-Modified Soil from the Ottawa Area in the Laboratory. MTC Report EM-42, 1980.
8. J.C. Jacobs. The Road Research Laboratory Frost Heave Test. G.B. Dep. Sci. Ind. Res., TRRL Note LN/766/JCJ, 1965.

Publication of this paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization.

Improvement of a Substandard Base Aggregate with Lime

PHILIP A. SEDDON AND DHANESH B. BHINDI

High inflation in road construction and rehabilitation costs, particularly in the area of haulage, has caused the New Zealand National Roads Board, through its Road Research Unit, to look for economies. One solution has been to use lime as an additive to base aggregates that would not normally comply with the demanding specification for this material. In some instances this could mean using a substandard base aggregate from a local quarry instead of an up-to-standard material hauled from a quarry farther away. In others, it could mean adding lime to the in situ recycling of a badly degraded base instead of using a granular overlay that has implied higher haulage costs. A description is given of the application of varying proportions of lime to a very substandard base aggregate, its testing in the laboratory, and full-scale field trials carried out at the University of Canterbury's test track at Christchurch. The field trials revealed that a substantial improvement in performance was obtained by adding 4 percent hydrated lime. This figure was predicted in the laboratory by the pH test, unconfined compression, double-punch tensile test, and the California Bearing Ratio. It is concluded that all these tests indicate the right concentration of lime, but it is not suggested that any one, on its own, is sufficient owing to the complex mechanisms at work in a road pavement.

A highway network of 96 000 km supported by a population of 3 million means that New Zealand has always had to spread its road funds thinly. Beginning in 1974, serious inflation cut the purchasing power of the road budget almost in half; and because a large part of the budget was for fuel, which must be imported, the haulage aspects of road construction and maintenance were the most seriously affected.

This has resulted in a form of a construction comprised of 100 to 150 mm of carefully specified unbound granular base supported as necessary on an unbound subbase and surfaced with a one- or two-coat surface treatment. This construction has served the country well and the little asphalt concrete used is mainly for arterial routes in and around the main population centers.

The New Zealand National Roads Board has, therefore, through its Road Construction Unit, sponsored considerable research on granular base material with a view to reducing costs and also avoiding further depletion of supplies of premium quality aggregate which are dwindling in some areas. Bartley (1) has summarized this work up to 1980, and the project described in the following paragraphs forms part of the ongoing program.

The use of lime, either as an additive to a local substandard aggregate or for in-situ stabilization of badly degraded pavement bases, has been found to provide substantial cost savings in some instances (2). Dunlop (3) has produced a definitive document on the use of lime in New Zealand road construction that covers laboratory tests, design, and construction methods. In the project under discussion here the lime treatment of a substandard aggregate has been examined and an attempt has been made to correlate laboratory test results with field performance under test track conditions.

LABORATORY WORK

The reactions between lime and fine-grained mineral particles have been categorized (4) in two phases: a cation exchange flocculation/agglomeration and a pozzolanic cementation. The extent to which these reactions take place depends on the nature of the fines, concentration of lime, time, and temperature; but there is no clearly defined changeover point. Dunlop (3) has outlined a number of tests suitable for determining the optimum proportion of lime. These test procedures were followed for determining the optimum proportion of lime to combine with a local substandard aggregate known as Teddington Greywacke and the results are discussed in the next section.

Teddington Greywacke

The weathered nature of the rock fragments and feldspars makes precise geological identification difficult and suggests a relatively advanced stage of weathering. The rock is of low metamorphic grade, probably of zeolite facies. As a base aggregate it fails the New Zealand specification (5) on four out of five counts:

1. It has a 10 percent fines crushing resistance of 90 kN compared with the required value of 130 kN when measured in accordance with the specification (similar to BS 812). Though not a part of the specification, its Los Angeles abrasion loss is 28 percent.
2. Its weathering resistance does not have the required quality index when tested according to the specification.
3. Its grading curve (percent passing) lies on the high side of the allowable envelope.
4. It has a sand equivalent of 20 compared with the 40 required by the specification.

Its use has been restricted to low-volume roads not requiring the National Roads Board (NRB) specification and unsurfaced roadways on private property. The various tests and subsequent field trials were carried out on mixtures of this aggregate, graded from 38 mm down, with 0, 1, 2, 3, 4, 5, and in some cases 6 percent by dry weight of hydrated lime. These are symbolized as TG0, TG1, TG2, TG3, and the like, in the remainder of the text. In some tests the larger sizes of aggregate had to be removed, and the proportion of lime was adjusted accordingly.

Dry Density

The dry density/water content curves were obtained by the New Zealand standard (6) vibrating hammer method on 150-mm diameter specimens. This method has been found to give densities close to those obtained in the field. The results shown in Figure 1 reveal that the effect of increasing lime content is to increase the optimum water content and lower the maximum dry density. This is in line with results for fine-grained soils (4), and may be attributed to the agglomeration of fine particles.

Conditioning Time

The effect of conditioning time on the development of shear strength was estimated by means of the falling cone test (7). The material passing the 4.75 mm sieve was mixed at optimum water content, cured at 20°C in sealed bags for varying intervals, and then given a standard compaction of six tamps at a pressure of 2.5 MPa in a kneading compactor. The results shown in Figure 2 show the desirable conditioning time to be between 10 and 50 hr depending on the lime content. It can also be noted that as the percentage of lime increases, the rate of gain and maximum gain increases. These higher gains are nearly all lost if the mixes are cured beyond the optimum times.

Plasticity

Plasticity characteristics of the material passing the 0.425 mm sieve with up to 18 percent lime were obtained in the usual manner. The natural material without lime gave a liquid limit of 25 and a plasticity index of 9. The New Zealand specification does not have a requirement on plasticity, but this material would not comply with ASTM D 694-71 for crushed stone for dry and water-bound Macadam base courses which has an upper limit of 6 for plasticity index. At 9 percent lime (corresponding to 1.5 percent in the total mix) the liquid limit had changed to 32 and the plasticity index had remained at 9. Higher concentrations of lime made no further change.

An increase in liquid limit as a result of adding lime seems rather strange, but reference to the United Soil Classification System plasticity chart suggests a change from CL to ML. However, a sedimentation analysis indicates that the fraction of clay (finer than 0.002 mm) is about 9 percent of the material passing the 0.425 mm sieve. So, beyond 9 percent lime, the plasticity tests are being influenced by the physical properties of the lime itself.

Linear Shrinkage

A shrinkage test on the fraction passing the 0.425 mm sieve showed that 2 percent of lime reduced the linear shrinkage to 3 percent, after which higher lime concentrations made no further difference.

Acidity

The 1-hour pH test described by Eades and Grim (8) has been found to be a simple and reliable indicator of optimum lime content for stabilization in New Zealand. The pH of the untreated material was 7.4 and the addition of 3.5 percent of lime to the whole fraction raised it to 12.4, the pH of saturated lime water. This proportion of 3.5 percent could be taken as optimum, but the pH level rises quickly when small quantities of lime are added. The pH value was actually more than 12 at 1 percent lime, so 3.5 percent would probably be more than sufficient to bring about any desirable modification of fines.

Unconfined Compression Strength

For lime to increase the strength of 76 x 38 mm diameter cylindrical specimens in unconfined compression, some of the pozzolanic reactions, which form calcium alumino hydrate and calcium silica hydrate, must take place. Some carbonate may also be formed but these are not considered desirable (4).

Specimens were prepared from the fraction passing the 4.75 mm sieve with varying proportions of lime, compacted at optimum water content, and cured in sealed bags for 48 hr at 20°C. Without lime, the unconfined compression strength was 0.6 MPa which increased to 2.0 MPa at 3 percent lime. The maximum strength of 2.5 MPa was obtained at a lime concentration for the whole fraction of 4 percent.

Figure 1. Dry density-water content relationships.

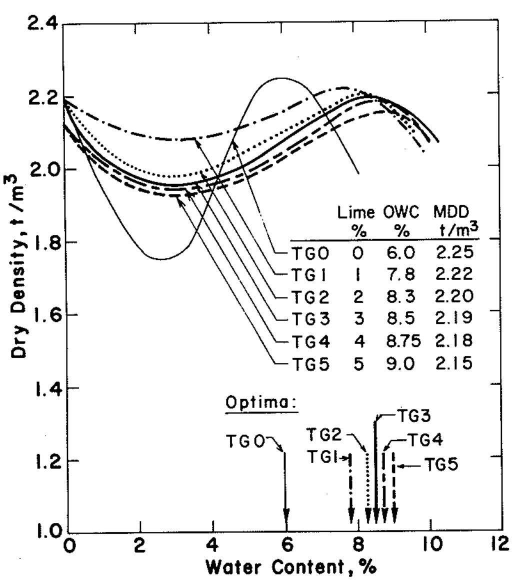
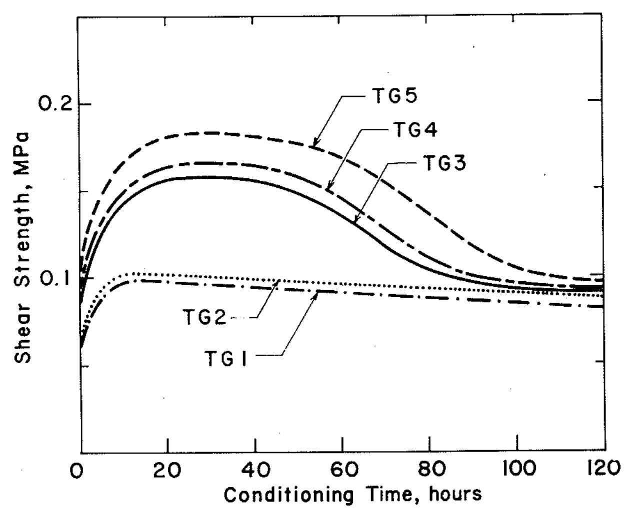


Figure 2. Variation of shear strength with conditioning time.



California Bearing Ratio

The CBR test is one of the most frequently used methods of assessing the strength of subgrades, subbases, and bases for thickness design of flexible pavements. Specimens were prepared from the whole fraction at optimum water content, cured for 24 hr, compacted, and soaked for 4 days before testing in the usual manner. The compaction was accomplished by vibrating hammer (6) which has been found to match constructed densities at the test track quite well.

Without lime, the soaked CBR was 15 rising to 40 at 3 percent lime and to a maximum of 53 at 6 percent lime. One percent lime eliminated the slight swelling found in the untreated material. These CBR values are very low for a base aggregate. Although CBR does not appear in the New Zealand specification for base aggregate (5), material complying in other respects would normally be expected to have a CBR in excess of 80.

Tensile Strength

The existence of pozzolanic cementation in a lime/aggregate mixture can be demonstrated by the existence of tensile strength. Dunlop (3) recommended the double punch test for determining tensile strength and a value of 80 kPa as indicating the boundary between the modification and cemented phases.

These tests were carried out on the whole aggregate fraction, precured for 24 hr, compacted with the vibrating hammer at optimum water content in CBR molds, and cured for 2 weeks at 20°C in sealed bags. Without lime, the tensile strength was 62 kPa, reaching Dunlop's critical 80 kPa at 4 percent lime and 120 kPa at 6 percent lime. According to Dunlop, therefore, the limiting lime concentration for cementation to have taken place was 4 percent.

FIELD TRIALS

The field trials were carried out at the University of Canterbury's test track which has been described in detail by Williman and Paterson (9). Briefly, two test vehicles consisting of twin 9.00 x 20 truck wheels, loaded to 40 kN, travel round a circular track of mean radius 9.2 m. An inching motor allows the vehicles to be moved slowly backward and forward for Benkelman beam tests.

An eccentric central bearing causes each vehicle to follow a curtate epicycloid and traffic an annular path approximately 1.25 m wide. The lateral distribution of wheel passes of the vehicles is approximately sinusoidal which is a reasonable simulation of that found on a public highway. At any point on the track, succeeding vehicle paths are distributed laterally in a pseudo-random fashion, but the pattern is repeated every 96 revolutions of the machine.

One vehicle has a chain drive to the wheels from an 8 kW electric motor; the other is pushed by the radial arm. As the track is covered by a roof, the latter vehicle has a sprinkling system that distributes water over the track at a rate of 1.75 l/m^2 over a 5-min period once a day. This corresponds to the average annual rainfall of 640 mm for Christchurch.

Pavement Design

The layer thicknesses were designed for 100,000 equivalent (8200 kg) design axle loads (EDA) according to the New Zealand highway standard for flexible pavements with thin surfacings (10). This design is based on repetitions of subgrade strain which for 10^5 is 1.05×10^{-3} mm/mm. Because of the extremely stiff alluvial gravel subgrade at this site, an artificial subgrade was introduced which consisted of a 500-mm layer of a natural alluvial sand mixed with 5 percent by dry weight of rubber tire buffings. This material is a waste product of the tire retread industry.

The artificial subgrade responds in a manner similar to a soft clay with a highly reliable modulus of 25 MPa; but it does not accumulate permanent strains as a real soil would. The design for this experiment then becomes 350 mm of pit-run gravel subbase (63 mm maximum size), 135 mm of base, and a single coat surface treatment. For design purposes it was assumed that the 135 mm base would meet the standard required by the New Zealand specification (5).

Pavement Construction

At the start of this experiment, the subbase and artificial subgrade were in place from a previous test. All that was necessary was to clean and relevel the surface of the subbase.

The aggregate was delivered to the site via a weigh bridge in 7- to 8-tonne truck loads. The material was dumped on a 15 m^2 concrete mixing area, its water content taken, and spread out to a uniform thickness of about 150 mm. The calculated mass of hydrated lime was then spotted in bags over the aggregate. The bags were split open, the water content adjusted to optimum, and, using a light agricultural tractor with a loader bucket, the whole was mixed thoroughly until it has a uniform color and consistency. This machine was then used to place the mixes with 0 through 6 percent lime in the test track building. The mixes were leveled and screeded to the desired surcharge by hand.

As it was winter, curing took longer than in the laboratory tests. Initial compaction was accomplished by a vibrating footpath roller, followed by trimming, and final rolling was with a 10-tonne, smooth-wheel roller. The lime-stabilized material was found to adhere to the steel wheels of the 10-tonne roller necessitating the use of the water sprinklers. This resulted in the base becoming much wetter than desired and, being winter, very slow to dry out. In the end, a 40 kW heater was used to dry the surface to a condition suitable for surface treatment.

A hard crust formed on the base, possibly containing calcium carbonate, that was laboriously removed before applying the surface treatment. In spite of this, the base was found to be virtually impervious, and the hot bitumen hardly penetrated it. The full pavement was constructed to a width of 3 m, and each of the seven sections was approximately 8 m long.

Before the surface treatment, small excavations were made in each section to the top of the subbase and subgrade. Steel plates 100 x 100 x 5 mm were placed at the centerline on the surface of the subbase and subgrade, and a 5-mm diameter steel rod was fixed in the center of the plate perpendicular to the surface. These rods were just long enough to reach to about 10 mm below the finished surface. They were enclosed in a plastic tube and plugged to prevent ingress of grit and bitumen. The base and subbase that had been removed were replaced and compacted so the material just filled the hole. This system has been found quite successful in obtaining transient and permanent displacements at depths within pavements constructed at the test track.

The machine is run 24 hr a day and various tests are carried out at intervals varying from 1,000 wheel passes at the beginning to 100,000 passes toward the end of an experiment.

In Situ Density

Density of the base sections was measured after construction and at intervals until trafficking was completed at 550,000 EDA. A Troxler nuclear densimeter was used, and it gave densities up to 100 kg/m^3 lower and water contents up to 2 percent higher than the New Zealand standard balloon densimeter (6). The results were, therefore, adjusted to the latter values.

The results at the beginning and end of the trafficking are given in Table 1. The field compaction gave values between 98 and 103 percent of the laboratory-determined maximum dry densities and the generally high performance of these pavement sections can, in part, be attributed to these high initial densities. It can be seen from the table that the TG0 had the highest initial density (and lowest voids) which increased with trafficking. Water contents fell slightly, but all sections were virtually saturated for the whole of the traffic loading.

Permanent Deformation

After construction, concrete pads were established on either side of the trafficked path, clear of any possible influence of pavement movement, at three places on each section. A straightedge could then be placed on these pads and depths to the surface measured at fixed distances from either side of the centerline to obtain cross sections. These were obtained at frequent intervals during the traffic loading. The average permanent surface deformations at the end of the traffic loading are given in Table 2. It can be seen that the untreated material, TG0, had the highest deformation and TG2, TG4, and TG5 the lowest. TG3 and TG6 were influenced by local distressed areas.

These results do not reveal much as they are considerably scattered and influenced by local trouble spots that had to be repaired to keep the vehicles running. Perhaps more revealing is that the maximum depression from a 2 m straightedge exceeded 12 mm in sections TG0, TG1, TG2, and TG3 but were nowhere near this in TG4, TG5, and TG6. A depression of 12 mm from a 2 m straightedge is considered by Croney (11) to indicate a critical condition. Permanent displacements at the top of the subbase and subgrade were negligible.

Pavement Deflection

Pavement deflection was measured on the surface between the truck wheels (loaded to 40 kN) by a Benkelman beam using the recovery method. In each section, three points were marked on the centerline, two of these were at the buried probes. Deflections were measured on the surface and on the probes, directly between the wheels and when the wheels were 600 mm away. This gave six positions of vertical deflection for computer modeling discussed in the next section.

Deflections were generally found to remain consistent over the trafficking period, but TG0, TG1, and TG2 showed some increase after 350,000 EDA. The mean deflections for the different materials are given in Table 3. The influence of lime content on surface deflection is apparent and is further illustrated in Figure 3. The reduction in deflection by the addition of up to 4 percent lime clearly suggests the development of stiffness by cementation processes.

Computer Modeling

The elastic layer computer program, ELSYM5, (12) and the finite element computer program, CANTYROAD, (13) were used to model vertical deflection at the six points. The CANTYROAD program gave slightly better correlation (maximum difference from measured deflection 0.08 mm) and indicates the following moduli:

Subgrade	25 MPa
Subbase (lower half)	65 MPa
Subbase (upper half)	170 MPa
Base (no lime)	385 MPa
Base (1-3 percent lime)	465 MPa
Base (4-6 percent lime)	550 MPa

The modeling is not particularly sensitive to Poisson's ratio and appears to lie in the range of 0.35 to 0.40. The CANTYROAD program also indicated a subgrade strain of 1.06 - 1.08 x 10^{-3} mm/mm compared with 1.05 x 10^{-3} used in the design.

Particle Size Analysis

Samples of each material were taken on delivery, after mixing and construction in the pavement, and from the center of the track on completion of trafficking. Grading curves were then obtained by a wet sieve analysis according to New Zealand standard (6). Space limits presentation of all the data but two sets of grading curves for the TG0 and TG4 in Figures 4 and 5 illustrate the pattern. Because the TG0 material has such a low crushing strength, it exhibits considerable breakdown between stockpile and pavement and further breakdown under traffic loading, particularly of the larger particles.

The grain-size distribution shown in Figure 5 for TG4 is typical of the higher lime concentrations and shows a significant reduction in fines between stockpile and pavement and little effect from traffic loading. There is an increase in the material retained on the larger sieves (19 and 9.5 mm) indicating the bonding of some fines to larger particles as time progresses. Consideration of all the grad-

Table 1. Dry density parameters at start and completion of traffic loading.

Condition	Traffic (10^3 EDA)	Material TG0	TG1	TG2	TG3	TG4	TG5	TG6
Dry density	0.5	2288	2216	2204	2157	2140	2139	2170
(kg/m^3)	550	2346	2280	2240	2238	2203	2142	2218
Total voids	0.5	13.7	16.4	16.8	18.6	19.2	19.3	18.1
(%)	550	11.5	14.0	15.5	15.5	16.9	19.2	16.3
Water content	0.5	6.0	7.2	7.0	8.3	9.2	8.8	9.3
(%)	550	4.9	6.2	6.6	7.1	8.0	8.5	8.1
Saturation	0.5	100	97	92	96	100	98	100
(%)	550	100	100	96	100	100	95	100

Table 2. Change in surface deformation with traffic loading.

Traffic (10^3 EDA)	Average Surface Deformation (mm) Test Strip TG0	TG1	TG2	TG3	TG4	TG5	TG6
0.5	0	0	0	0	0	0	0
20	1.50	1.00	1.00	1.50	0.50	0.50	0.50
50	1.75	1.00	1.50	1.75	1.00	1.50	1.25
150	2.75	1.50	1.75	2.50	1.75	2.25	2.25
250	3.75	2.50	2.25	3.50	2.50	3.25	3.25
350	4.50	3.25	3.00	4.00	3.25	3.50	4.00
450	5.50	4.75	3.50	5.75	4.50	5.00	5.00
550	7.00	6.25	4.50	6.75[a]	5.50	5.50	6.50[a]

[a]Included local distressed area.

Table 3. Transient deflections for different lime contents.

Probe Position Horizontal	Vertical	Average Surface Deflection (mm) Test Strip TG0	TG1	TG2	TG3	TG4	TG5	TG6
Between wheels	On surface	0.86	0.78	0.81	0.78	0.75	0.75	0.73
	On subbase	0.75	0.72	0.67	0.69	0.69	0.69	0.71
	On subgrade	0.47	0.43	0.50	0.42	0.46	0.50	0.50
600 mm from wheels	On surface	0.10	0.10	0.10	0.11	0.11	0.11	0.12
	On subbase	0.12	0.11	0.10	0.11	0.13	0.12	0.13
	On subgrade	0.12	0.11	0.11	0.11	0.11	0.11	0.14

Figure 3. Effect of lime content on surface deflection.

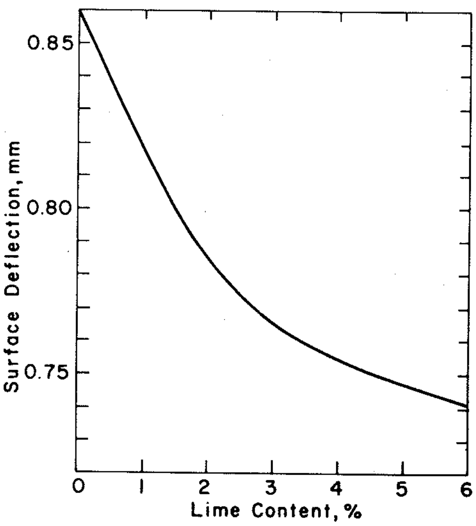


Figure 5. Changes in grading for TG4.

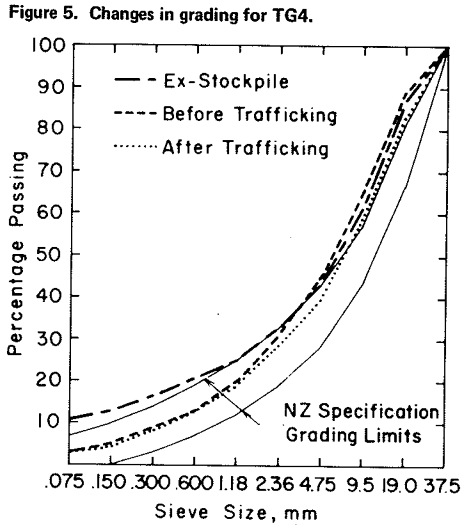
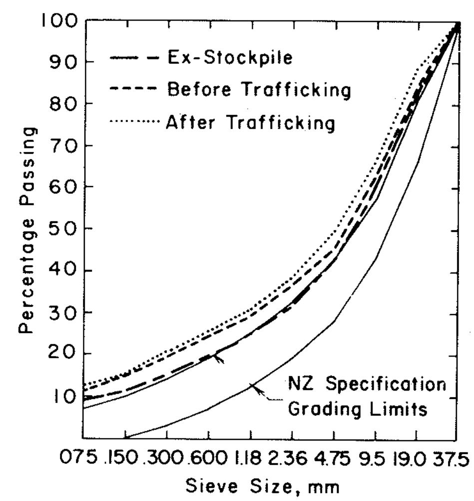


Figure 4. Changes in grading for TG0.



ing curves suggests that agglomeration begins at 2 percent lime and after 4 percent no further change takes place.

If the grading curves are plotted to a log-log scale and approximate a straight line, then the gradient of this line is the exponent 'n' in the idealized grading curve of Talbot and Richart (14):

$$Pd = (d)^n/Dm$$

where

Pd = the proportion passing sieve size d
Dm = the maximum particle size, and
n = a fraction lying between 0.35 and 0.45 for maximum density.

Grading curves were plotted for each material and the gradients of the lines obtained by linear regression. Two examples, for TG0 and TG1, are given in Figures 6 and 7. A plot of the 'n' values in the pavement against percentage of lime is shown in

Figure 6. Changes in grading exponent for TG0.

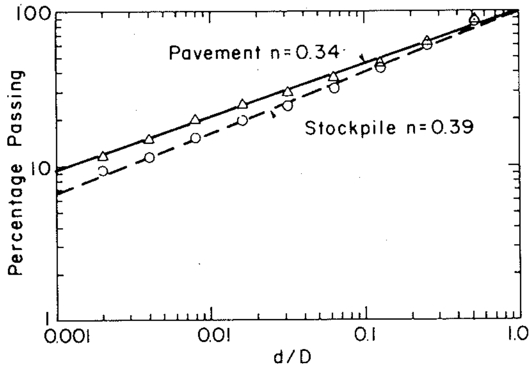


Figure 7. Changes in grading exponent for TG4.

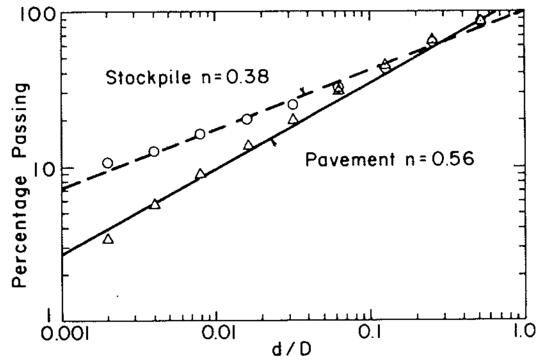


Figure 8. This shows that agglomeration of particles begins above 1 percent lime and reaches a maximum at 3 percent.

Surface Cracking

The condition of the pavement was observed continuously from the end of construction to the end of trafficking. Conditions were recorded on specially designed, condition survey forms; photographic records were also kept. At approximately 75,000 EDA, the first hairline cracks (visible from 400 mm) appeared, mainly in sections TG0, TG1, TG2, and TG3. The cracks were short, randomly scattered and soon disappeared. It is believed that they were associated with drying out of the base materials. At 200,000 EDA further cracks appeared in these same sections concentrated near the edges of the trafficked path. These were considered to be associated with permanent deformation and, once formed, allowed further drying and shrinkage to take place.

The TG0, TG1, TG2, and TG3 sections started developing extensive surface cracking after 500,000 EDA; this was associated with increased deflection and rutting. The cracks were between 0.1 and 0.3 mm wide, visible in natural light from 2.5 m, and were mainly transverse to the direction of traffic but developed in an alligator pattern in TG0 and TG1. There was no cracking evident in the TG4, TG5, and TG6 sections; Figure 9 shows the relationship between crack length per unit area and lime content.

CONCLUSIONS

In road construction, lime is most frequently applied to soft clay subgrades because the increased strength enables a thinner pavement to be used. This experiment has shown that lime can be usefully applied to a substandard base aggregate to improve its performance. The laboratory tests and field results are summarized and compared in the following sections.

Laboratory Evaluation

It was found that the addition of lime to a graded aggregate increased the optimum water content and lowered the maximum dry density under standard compaction. Optimum conditioning time (at 20°C) was between 10 and 50 hr at all lime concentrations. Excessive conditioning reduced strength. The addition of lime to the fines increased the liquid and plastic limits and slightly reduced the plasticity index. At the higher concentrations, the physical properties of the lime were influencing the plasticity characteristics. Two percent lime caused a reduction in the linear shrinkage of the fines.

The Eades and Grim pH test indicated an optimum lime content of 3.5 percent, but even at 1 percent, the effect was becoming apparent. The unconfined compression strength showed a significant increase with 1 percent lime and reached a maximum at 4 percent lime. Soaked CBR increased from 15 without

Figure 8. Effect of lime content on grading exponent.

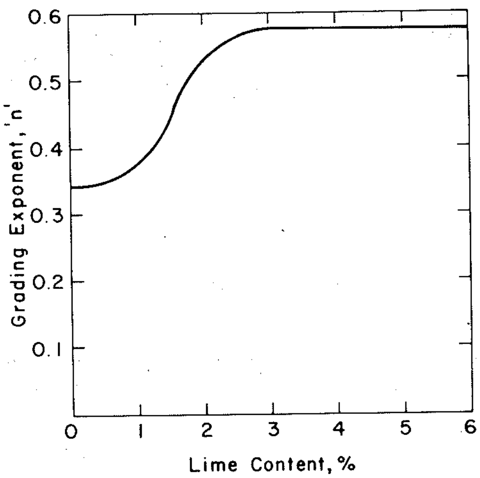
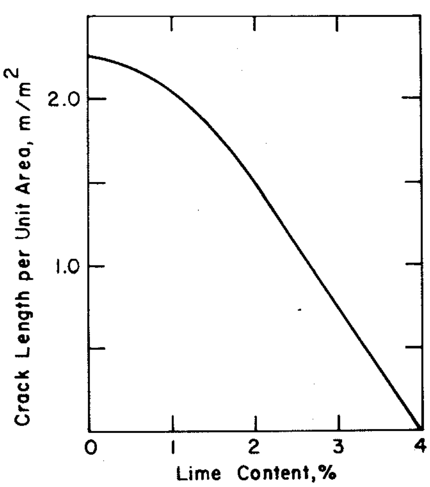


Figure 9. Effect of lime content on cracking.



lime to 53 at 6 percent lime, 4 percent being an optimum value. Four percent lime was required to increase the tensile strength to 80 kPa, the value suggested (3) as marking the boundary between the modification and cemented phases of lime treatment.

Field Evaluation

Although the response of sand/rubber subgrade to wheel loads is similar to that of soft clay, it suffers practically no permanent deformation. Previous trafficking on the subbase had resulted in that material being completely compacted and exhibiting virtually no permanent deformation in the test. The results, therefore, refer strictly to the performance of the bases. A high degree of compaction in the field gave densities for all sections of about 100 percent of the theoretical maxima. The untreated section had the lowest voids content at the start and completion of trafficking and all sections were virtually saturated throughout. Saturated bases cause considerable concern in New Zealand because of the loss of stiffness and pore pressure buildup particularly in cases where mechanical and hydrothermal degradation has created clay fines.

In this test hydrothermal degradation which may take several years was not present, but it is believed that the presence of lime would have helped had it been so. Rutting (being restricted to the base layer) was generally low because of the high initial densities but deformations from a 2 m straightedge were higher in the 0-3 percent lime sections than the others. Transient deflections were reduced with increasing lime content, the first 4 percent causing the greatest reduction.

Computer modeling indicated that the untreated base had a modulus of 385 MPa; 1, 2, and 3 percent had an average of 465 MPa; and 4, 5, and 6 percent had an average of 550 MPa. For the untreated section, there was a significant degradation of the aggregate between stockpile and constructed pavement with a further degradation under traffic loading, particularly of material retained on the 19 mm sieve. Three percent lime shows a significant reduction in the material finer than 4.75 mm but still with an increase in the proportion passing the 19 mm sieve. Four percent and more continues this pattern except that the proportion passing the 19 mm sieve is held constant. The effect on the idealized grading exponent 'n' of Talbot and Richart (14) is clearly demonstrated in Figure 8. This change in grading is reflected in the lower dry densities with increased lime contents.

The significant surface cracking apparent in the 0, 1, 2, and 3 percent lime sections after 500,000 wheel passes is attributed to fatigue from the larger number of wheel passes. The greater stiffness developed by 4 percent or more lime is sufficient to reduce this.

Correlation of Field Results with Laboratory Tests

The significant improvement in field performance (by the criteria of permanent deformation, deflection, particle degradation, and surface cracking) by the addition of 4 percent hydrated lime indicates that 4 percent is the optimum value. However, this should be compared with the laboratory obtained optima:

Test	Lime (%)
Eades and Grim pH test	3.5
Unconfined compression test	3-4
Double punch tensile test	4
California Bearing Ratio	4

All four tests appear to indicate the best concentration of lime, and it would appear that any one of them may be sufficient for practical purposes. Possibly the pH test would be the most practical because it is simplest and quickest; but it gives no indication of strength gain. There are many complex mechanisms at work in a road pavement under a rolling wheel load, and it is recommended that materials be tested by at least three of these methods. If one were to be left out, it could be either the unconfined compression or the tensile test.

ACKNOWLEDGMENT

We acknowledge with gratitude the financial sponsorship of the work described in this report by the National Roads Board of New Zealand. We are also extremely appreciative of the consciencious efforts devoted to this project by Neil W. McDougall.

REFERENCES

1. F.G. Bartley. A Review of Aggregate Research in New Zealand. RRU Bull. 50, National Roads Board, Wellington, 1980.
2. National Roads Board (NZ). Stabilisation Seminar: Papers and Report. RRU Bull. 40, National Roads Board, Wellington, 1978.
3. R.J. Dunlop. Lime Stabilisation for New Zealand Roads. RRU Tech. Recommendation TR/2, National Roads Board, Wellington, 1977.
4. TRB Committee A2J03. State of the Art: Lime Stabilization. TRB, Transportation Research Circular 180, 1976.
5. National Roads Board (NZ). Specification for Crushed Basecourse Aggregate, NRB M/4. National Roads Board, Wellington, 1981.
6. SANZ. Soil Tests for Engineering Purposes, NZS 4402, Parts I and II. Standards Assn. of New Zealand, Wellington, 1980/81.
7. S. Hansbo. A New Approach to the Determination of the Shear Strength of Clays by the Fall Cone Test. Proc., Swedish Geotechnical Institute No. 14, Stockholm, 1957.
8. J.L. Eades and R.E. Grim. A Quick Test to Determine Lime Requirements for Soil Stabilization. HRB, Highway Research Record 139, 1966.
9. A. Williman and W.D.O. Paterson. A Track for Accelerated Testing of Highway Pavements. N.Z. Engineering, 26(3), March 1971, pp. 73-77.
10. National Roads Board (NZ). Pavement Design, Highway Standards S/4. National Roads Board, Wellington, 1974.
11. D. Croney. The Design and Performance of Road Pavements. HMSO, London, 1977.
12. G. Ahlborn. Elastic Layered Systems with Normal Loads. ITTE, Univ. of California, Berkeley, 1972.
13. C.N. McKenzie and A.J. Carr. A Finite Element Analysis Program for Multilayer Pavement Structures. Computer Program Library, Civil Engineering Department, Univ. of Canterbury, Christchurch, 1981.
14. A.N. Talbot and F.E. Richart. The Strength of Concrete, Its Relation to Cement Aggregates and Water. University of Illinois, Engineering Experimental Station, Bulletin No. 137, 1923.

Publication of this paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization.