Design, Construction, and Frost Susceptibility of Lime Stabilized Marine Clay in Highway Subgrade Fill

A.M. BATTEN AND A.J. HANKS

An extremely, wet, soft marine clay was modified with a small percentage of hydrated high-calcium lime and subsequently placed in freeway subgrade fills in the Ottawa area (1). After some adjustment of the construction equipment and operations, the contractor succeeded in achieving high-quality, well-controlled fill construction. The lime treatment reduced construction costs by reducing the requirements to dispose of the soft plastic clay and the quantity of imported fill required to construct the fill sections. In the winter and spring following completion, the pavement was opened to traffic, and severe differential frost heaving occurred. Traffic lanes had to be closed and traffic was confined to one lane on the passing side of each of the divided roadways. After the spring thaw, substantial corrective and preventive work was required to restore the roadway.

Outlined in this paper are the laboratory test procedures to determine the stabilizing effects of the lime on the soft clay, the construction procedures used and conditions that existed during the treatment and cut and fill operations, and subsequent site and laboratory investigation to determine frost heaving of soft clay treated with various percentages of high-calcium lime. Also outlined are the remedial measures taken to rehabilitate the roadway.

REFERENCES


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simulates practical construction procedures of mixing the lime into the clay and allowing curing or strengthening of the material before removing and placing in fills. The compacted CBR specimens were soaked for 4 days before penetration testing was performed. Approximate data collected from the CBR tests are given in Table 1 and the penetration curves are shown in Figure 1.

Atterberg Limits

Details of this test procedure may be found in the Ontario Ministry of Transportation and Communications (MTO) Testing Manual (4,5).

The addition of lime has a marked effect on the Atterberg Limits. Both the liquid limit and the plastic limit increase with increasing lime content. However, the plastic limit increases more than the liquid limit; hence, the plastic index or range of plasticity is decreased.

The tests were performed by adding small percentages of lime to specimens of the natural soil in increments of 1 to 2 percent. The lime-soil mixture was cured for 24 hr in plastic bags before the normal test procedures were performed. The plastic limit increases with the addition of lime to a point where further increments show little change. This is known as the fixation point, Hilt and Davidson (5). For the material tested, the fixation point was found to be approximately 1.8 percent for both high-calcium and dolomitic lime (Figure 2).

Grain-Size Distribution

The grain-size analysis tests on the lime/clay mixtures cured for 24 hr showed a coarsening effect as the percentage of lime was increased. This is because the small clay-size particles agglomerate to form silt and sand-size particles thus changing the apparent particle-size distribution of the soil mass. Figure 3 illustrates the grain-size distribution of the natural clay and lime/clay mixtures after curing 24 hr.

This test is used to identify the frost susceptibility of untreated soils for design purposes. MTC criteria for identifying frost susceptibility of a soil mass are as follows:

Grain Size Between 5 μm and 75 μm (%) Frost Susceptibility
5-40 Low
40-55 Moderate
55-100 High

Both the natural and treated soils indicated low frost-susceptibility characteristics by these time-tested criteria.

CONSTRUCTION

The successful contractor's bid indicated that approximately $200,000 could be saved by treating the soft clay with 2 percent lime and placing it in the fills rather than wasting it.

Construction was started in the fall of 1976 but was primarily carried out during the summer months of the following year. The contractor hauled the lime from the supply source in large truck-mounted tanks with rear cyclone-spreaders attached. A self-powered pulvi-mixer was brought to the site to mix in the material. Both of these units bogged down in the underlying soft clay and progress was impeded to

Table 1. CBR design data.

<table>
<thead>
<tr>
<th>Property</th>
<th>Lime Content</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Natural</td>
</tr>
<tr>
<td></td>
<td>Material</td>
</tr>
<tr>
<td>Water content (%)</td>
<td>49</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>21</td>
</tr>
<tr>
<td>Dry density (t/m³)</td>
<td>1.14</td>
</tr>
<tr>
<td>CBR</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Figure 1. CBR test values of lime/clay mixtures after curing 24 hr.

Figure 2. Plastic-limit test values with high-calcium and dolomitic lime mixtures after curing 24 hr.
Figure 3. Grain-size distribution of high-calcium lime/clay mixtures after curing 24 hr.

Figure 4. Excavation of lime-treated clay.

Figure 5. Disc mixing lime into clay.

the point that alternative equipment was brought to the site.

The lime was applied with 15 m³ self-propelled, rubber-tired earth scrapers (Figure 4). After a few trial runs the lime dispensed through a narrow discharge opening over the scraped width was spread uniformly while the scraper traveled at a uniform speed (approximately 15 km/hr). After a short waiting period, the material was mixed into the clay with a 90-cm diameter, tractor-hauled disc. The disc was set at a slight angle during the first pass and as stability improved the angle was increased to facilitate mixing of the lime and clay at a greater depth (Figure 5). This process produced a uniform mixture and broke down the clods to a clayey, sand-like texture after four to five passes. The treated depth of loose material was approximately 40 cm.

After mixing, the material was allowed to cure and strengthen until field observations indicated that it could be excavated and compacted in fills with conventional equipment. The curing usually took 6 to 8 hr. The treated material was then excavated with scrapers and placed in compacted lifts of 15 to 25 cm. A 30-cm depth of material was removed, leaving 10 cm of treated material to support the scraper for the subsequent treatment layer. The fill lift was compacted with a 25 t self-propelled segmented drum, a Pack-All Unit (Figure 6). The lime-soil mixture demonstrated remarkable stability under the heavy compaction unit.

Laboratory Target Density

Optimum standards for soil used for fill as established under laboratory conditions by the standard proctor test are as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum wet density (t/m³)</td>
<td>2.04</td>
</tr>
<tr>
<td>Maximum dry density (t/m³)</td>
<td>1.69</td>
</tr>
<tr>
<td>Optimum moisture content (%)</td>
<td>21</td>
</tr>
</tbody>
</table>

Data from Site Tests

The following compaction data were obtained from continuous testing of the fill operation.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field wet density (t/m³)</td>
<td>1.80 to 2.01</td>
</tr>
<tr>
<td>Field dry density (t/m³)</td>
<td>1.53 to 1.69</td>
</tr>
<tr>
<td>Field moisture content (%)</td>
<td>16 to 23</td>
</tr>
<tr>
<td>Percent compaction (%)</td>
<td>91 to 100</td>
</tr>
</tbody>
</table>
Most of the density tests ranged close to 95 percent of the target value. Density tests of the in situ clay indicated 1.64 to 1.87 t/m³ wet density and 1.29 to 1.45 t/m³ dry density with 30 to 50 percent moisture content. This indicates an approximate 15 percent densification of the solids. Moisture loss during the treatment and grading ranged from 10 to 25 percent.

This degree of densification was not expected and densification was not estimated during the design. It may be that the moisture loss during treatment and subsequent grading exceeded moisture loss in the laboratory during mixing and preparation of specimens. Sunny, warm, and dry weather prevailed during most of the operation; but stabilization was impeded during the periods when the weather was cool and wet. When the temperature was below 5°C, progress was slowed to the point that the contractor suspended operations.

The grading operation produced stable, uniform, homogeneous subgrade conditions. The subgrade treated with 5 percent lime in the cut (45 cm thickness) was quite stable. This section was insulated before applying the granular base because of the potential differential frost action within the soft clay under the treated material.

Granular base and paving was completed under a follow-up project during 1978. The granular base was 15 cm thick and consisted of well-graded, < 22.4 mm nominal-size material. Bituminous pavement was 28 cm deep and consisted of dense-graded hot mix which met the standard Marshall mix requirements specified by MTC. Paving was completed in the early fall and the roadway was opened to traffic. The paving was of a high quality and provided an excellent ride when the roadway was opened.

PERFORMANCE

Excellent pavement conditions continued until substantial below-freezing weather occurred. Near the end of December 1978, some differential frost heaving was noted on the lime-treated fill sections. These conditions seemed to stabilize after December and the pavement remained slightly uneven through January and February 1979. Continuous cold weather prevailed through this period. In the first week of March, during a sudden warm period accompanied by rainfall, substantial differential frost heave distortion occurred over a 2-day period. Heave differentials as high as 20 cm developed in the driving lane over lengths of 6 to 7 m (Figure 7). The distortions were mainly located along the shoulder and outer pavement edge, extending into the midlane approximately 1.5 m from the edge of the pavement (Figure 8). Large cracks developed in the gravel surface of the shoulders. It was possible to divert traffic to the passing lane while the driving lane was barricaded because the passing lane experienced considerably less distortion. This condition lasted for 2 months during major rehabilitation work before normal traffic operations could be restored. The frost heave distortions did not occur in the cut sections with insulated subgrade.

Weather

Near-continuous freezing weather began about November 16, 1978. During December 1978 and January-February 1979, persistent cold weather conditions prevailed. From November 16, 1978, to February 28, 1979, the freezing index totaled 1042.3 Celsius degree-days. A relatively small amount of freezing occurred during March.

Table 2 outlines the freezing index and precipitation during the period of frost heave occurrence.

<table>
<thead>
<tr>
<th>Data</th>
<th>Freezing Index (Celsius degree-days)</th>
<th>Precipitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>November 1978</td>
<td>72.4</td>
<td>47.1</td>
</tr>
<tr>
<td>December 1978</td>
<td>251.7</td>
<td>26.2</td>
</tr>
<tr>
<td>January 1979</td>
<td>316.4</td>
<td>19.9</td>
</tr>
<tr>
<td>February 1979</td>
<td>401.8</td>
<td>25.6</td>
</tr>
<tr>
<td>March 1979</td>
<td>3.2</td>
<td>39.6</td>
</tr>
</tbody>
</table>
Figure 9 illustrates the daily mean temperatures from November 1978 through March 1979.

FROST HEAVE, SITE INVESTIGATION

Test pits at the pavement edge and within the shoulder area indicated substantial ice lens formation in the lime-treated subsoil. The lenses varied in thickness from 0.5 to 4 cm and were located at various 2 to 15 cm intervals to depths of 1.5 to 1.8 m below the roadway surface.

Samples of the treated subgrade material were obtained. In addition, samples of the soft, untreated clay were obtained for follow-up laboratory test and evaluation.

REMEDIAL MEASURES

Remedial measures to restore the roadway after the thaw period were installing subgrade drainage and sealing the shoulder surfaces by asphalt paving. Subdrains were installed along the edges of pavement 1.2 to 1.5 m below the surface. The drains were 100-mm diameter, slotted plastic pipe wrapped with a geotextile filter fabric. The drains were placed in 300-mm wide trenches bedded and backfilled with a clean, fine sand. The sand backfill contained 2 to 4 percent passing the 75 mm sieve. Outlets to the roadside ditches were spaced at approximately 100 m intervals.

During warm weather in late May, the distorted asphalt-pavement surface was rolled with a 25 t self-propelled steel roller. After two to three passes, much of the heave distortion was reduced; however, the surface was still distorted from its original condition, and the ride over the distorted areas was quite rough. The rolling treatment was followed by milling an average 40 mm depth from the distorted sections. The areas were then resurfaced flush with the adjacent undistorted pavement. The milling and repaving varied in width from 1 to 2.5 m from the pavement edge. In conjunction with this, the full shoulder widths were repaved over the entire length of the areas with treated subgrade fill material.

These procedures restored an almost even ride and reduced the magnitude of the frost heaving in the winters subsequent to the restoration. Nevertheless, frost heaving continues to be a problem in the fill sections, and additional preventive treatment will be required in conjunction with future overall rehabilitation. This will probably involve installing insulation which will be expensive.

LABORATORY INVESTIGATION

After review of the original test data and consultations with several engineers having considerable knowledge and background in frost action, it was decided to do a laboratory investigation (7) of mixing various percentages of lime with samples of the soft, silty clay from the project site (natural material). Samples of treated fill from the site were also tested.

Equipment

The frost heave apparatus was constructed at the Ontario Hydro Research Division laboratories. It consists of a box fabricated from 13-mm thick plywood and lined with 25-mm thick polyurethane foam sheet on the bottom and to a height of 190 mm on the sides. Figures 10 and 11 show details of the construction. A metal tray fits inside the foam lining and has square acrylic blocks supporting a wire-mesh plate. A number of Pyro-tenex heating cables are
strung the length of the tray between the acrylic blocks. Three additional layers of polyurethane foam are fitted on top of the tray, and 11 holes are made in a symmetrical pattern to accept the specimen containers.

The specimen containers are fabricated from acrylic tubing with 6-mm thick walls and are 200 mm long. A perforated acrylic baseplate fits snugly into the bottom of each specimen container. A disc of geotextile material is used to separate each specimen from the baseplate. Changes in height of the specimens are measured by a dial gauge mounted on a bar that rests on top of the acrylic cylinder.

The whole frost box assembly containing samples is placed in a freezer. Sufficient water is poured into the tray to keep the level approximately 25 mm above the bottom of the test specimens. The water is kept at approximately +4°C and the freezing temperature at approximately −17°C to maintain a one-directional temperature gradient across the specimens. Figure 12 shows the frost box containing specimens in the freezer chest.

The test specimens are compacted in a regular 0.944 dm³ Proctor mold and then extruded. Each specimen is wrapped in a layer of petroleum jelly-coated plastic wrap. The outside of the wrapped specimen is then coated with petroleum jelly and placed on the geotextile disc resting on the perforated acrylic baseplate. The acrylic cylinder is lowered over the specimen and the annular space between the specimen and cylinder is sealed with a thick bead of petroleum jelly, which is put in place by using a large plastic syringe. This is to prevent the specimen from freezing to the cylinder and thus causing inaccurate height-change measurements.

Test Program

Three bulk samples of material (referred to later as series 1, 2, and 3) were obtained from the site for testing: two in the natural state and one of lime-treated material from the fill area. The samples were enclosed in plastic bags inside normal sample bags to retain the natural water content.

For each natural material, duplicate specimens were compacted with high-calcium lime contents of 1, 2, 4, and 6 percent, based on dry mass. The procedure briefly outlined under design parameters was followed, with the exception that a Proctor mold was used. After compaction, the specimens were stored in plastic bags in a constant humidity environment for about 6 weeks to simulate field aging.

Series 1. The first natural material had a water content of 31.2 percent, and the molding data are given in Table 3.

Series 2. A set of three samples was prepared from the treated material, and the molding data are given in Table 4. The compaction and curing was the same for these specimens as for series 1.

Series 3. A third set of samples prepared from the second natural material had a water content of 68.9 percent, which was considerably higher than the natural material used for series 1. The molding data are given in Table 5.

Series 4. A fourth set of samples was prepared from the first natural material used for series 1 (w = 31.2 percent) before the frost heave tests. The ends of these specimens were immersed about 25 mm below the water level in the tray and allowed to absorb water for 24 hr before being placed in the
Table 3. Molding data (series 1).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Property</th>
<th>1 and 2</th>
<th>3 and 4</th>
<th>5 and 6</th>
<th>7 and 8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wet density (avg t/m³)</td>
<td>1.908</td>
<td>1.832</td>
<td>1.817</td>
<td>1.766</td>
</tr>
<tr>
<td></td>
<td>Water content (avg %)</td>
<td>30.5</td>
<td>32.2</td>
<td>31.5</td>
<td>26.9</td>
</tr>
<tr>
<td></td>
<td>Dry density (avg t/m³)</td>
<td>1.463</td>
<td>1.386</td>
<td>1.381</td>
<td>1.391</td>
</tr>
</tbody>
</table>

Table 4. Molding data (series 2).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Property</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wet density (t/m³)</td>
<td>2.019</td>
<td>2.019</td>
<td>2.019</td>
</tr>
<tr>
<td></td>
<td>Water content (%)</td>
<td>22.2</td>
<td>22.2</td>
<td>22.2</td>
</tr>
<tr>
<td></td>
<td>Dry density (t/m³)</td>
<td>1.652</td>
<td>1.652</td>
<td>1.652</td>
</tr>
</tbody>
</table>

Table 5. Molding data (series 3).

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Property</th>
<th>1 and 2</th>
<th>3 and 4</th>
<th>5 and 6</th>
<th>7 and 8</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wet density (avg t/m³)</td>
<td>1.654</td>
<td>1.654</td>
<td>1.673</td>
<td>1.692</td>
</tr>
<tr>
<td></td>
<td>Water content (avg %)</td>
<td>54.8</td>
<td>55.6</td>
<td>51.5</td>
<td>49.7</td>
</tr>
<tr>
<td></td>
<td>Dry density (avg t/m³)</td>
<td>1.069</td>
<td>1.061</td>
<td>1.104</td>
<td>1.130</td>
</tr>
</tbody>
</table>

Test Results

During the testing, dial gauge readings are taken of height changes of the specimens at convenient time intervals. These are plotted as heave in millimeters versus time in hours; a typical set of results for series 3 is shown in Figure 13. The hatched area between 12.7 and 17.8 mm represents the marginal frost-susceptibility classification according to the Transport and Road Research Laboratory (CRRL) in England. Readings below 12.7 mm are considered satisfactory and readings above 17.8 mm are considered unsatisfactory. All these ratings are taken at a 250 hr time-point for uniformity. Figure 14 shows a comparison of heaving for series 3 specimens with lime contents ranging from 1 to 6 percent, and Figure 15 shows heave in the natural soil samples. After a completed test, all specimens are photographed with a standard-size (110 mm high) compacted specimen which provides a visual comparison of the magnitude of heaving.

Table 6 gives the relative heaving in millimeters for the three materials. Some approximations were made for specimens where the heave was great. All the materials tested were in the CI-CH range, and the difference between the two natural materials was the water content. The material with the lower water content (31.2 percent) showed heaving magnitudes approximately half of those for the material with the higher water content (68.9 percent), except at 6 percent lime. This is confirmed to some extent by comparing the heaving for the material at a lower water content unsoaked and soaked before freezing.

The soaked samples showed a heaving magnitude approximately twice that of the unsoaked samples, except at 6 percent lime.

The natural material at a higher water content heaved less when 1 and 2 percent lime were added; and this is consistent with the change in grain-size distribution shown in Figure 3, which indicates that these materials become coarser with the addition of lime. This leads to an increase in permeability and probably a faster capillary rise in the specimens with 1 and 2 percent lime before freezing occurs. At higher lime contents, the increased bonding or cementing action of the lime with clay particles probably offsets the hydraulic pressures generated by freezing. In addition, the more open pore structure may assist in dissipating hydraulic pressures.

The field-treated specimens showed a magnitude of heave approximately the same as that for the natural material at the lower water content and 1 percent lime, which would indicate that the field lime content is of the same order.

It would appear from the limited testing carried out that the water content of the material before modification or stabilization is of great significance. Also, the addition of 1 or 2 percent lime, sufficient to increase the CBR of the material to a marked degree, is of little benefit if the material will be subjected to freezing. It is of some significance that all the specimens treated with 6 percent lime gave less than the 12.7 mm minimum readings recommended by the TRRL as being in the satisfactory range.

DISCUSSION

The laboratory frost heave testing and the performance of the project indicate that laboratory frost-susceptibility criteria for natural soils are not applicable to chemically treated materials. The 1 and 2 percent lime and clay mixtures appear to have drainage properties and capillarity similar to natural silt (5 to 75 mm grain-size soil). This phenomenon is probably due to cementation of the clay-size particles changing the grain size structure within the soil mass. The continued addition of lime appears to have increased the cementation process and changed the porosity. The reduced internal drainage reduces water access to the soil mass thereby reducing the main ingredient for ice lens formation. The increased strength from the added cementation further increases soil deformation resistances and further reduces the frost susceptibility of the soil mass.

The laboratory test procedures used in this project to evaluate the prepared lime/clay specimens and the sample of treated material from the project appear effective in simulating the environmental effects of moisture and frost on the chemically treated materials.

Because lime content is directly related to the frost susceptibility of lime/clay mixtures, close control of the lime content is of prime importance during construction. This should be done by con-

Table 6. Heaving magnitudes.

<table>
<thead>
<tr>
<th>Property</th>
<th>Lime Content</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
</tr>
<tr>
<td>First material (low w)</td>
<td>50</td>
</tr>
<tr>
<td>Field-treated material</td>
<td>58*</td>
</tr>
<tr>
<td>First material (soaked)</td>
<td>135</td>
</tr>
<tr>
<td>Second material (high w)</td>
<td>122</td>
</tr>
</tbody>
</table>

*aLime content assumed.
CONCLUSION AND RECOMMENDATIONS

1. The addition of small quantities of hydrated lime was most effective in stabilizing the soft clay material. The CBR and Atterberg Limits tests provided practical and accurate evaluation of the stabilization that occurred during construction. These tests should be continued for design evaluation in circumstances where stabilization of soft clay may be advantageous.

2. The grain-size distribution test of chemically treated specimens may provide an indication of stabilization; however, it should not be used alone to assess frost susceptibility.

3. Chemically treated soils should be tested to determine frost susceptibility. The samples should be tested in a freezing chamber with a continuous water supply. This test is not expensive and can be performed in a practical time frame.

4. Treatment of soft problem clays with various small proportions of lime should continue to be considered in areas affected by frost action. However, the aforementioned frost-heave test should always be part of the evaluation. Tight control of the lime content during construction must be adopted in any lime-treatment project.

REFERENCES

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 pozollanic 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 key parameter. 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 its Los Angeles abrasion loss is 28 percent.

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 re-


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