Evaluation of the Performance of Lime and Cement Treated Base Layers in Unpaved Roads

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ABSTRACT

Soil stabilization with cement and lime materials has been widely used to improve the mechanical properties of road subgrades and pavement layers, which consequently result in better performance, both in paved and unpaved roads. In this study, both laboratory experiments and finite element analysis were conducted to evaluate the performance of lime and cement treated base layers on soft clay subgrade. To achieve this, a number of mixtures with different lime and cement contents were defined and modelled, using finite element software, namely PLAXIS, as base layers under different loadings. The models were then analysed and compared in terms of vertical deformation under loads and the maximum applicable load. Analytical and numerical modelling of lime and cement treated soils require a number of soil parameters, which are usually obtained from expensive and time-consuming laboratory experiments. An alternative method was proposed in this study, in which the soil parameters required for the finite element analysis were obtained from the unconfined compressive strength tests, and estimated using the estimating functions available in available literature. The results of this study have showed that the application of the mixture with the highest modulus of elasticity for a base layer does not necessarily result in a pavement with the greatest bearing capacity. Rather, a correspondence should exist between the stiffness of the subgrade and that of the base layer.

KEYWORDS: Stabilization, Deformation, Finite Element Modelling, Cohesion, Internal friction angle.
INTRODUCTION

Soil stabilization refers to the process of changing soil physical and chemical properties to improve strength and durability. It has been used for many years to improve the characteristics of the subgrades with problematic soils. Pavements with stabilized base and subbase layers have been proven to deliver better performance during their service life [1]. Different soil types treated with various kinds and amounts of stabilizing agents have been investigated by many researchers over the years. For road pavements, cement and lime are among the main stabilizing agents. These materials are mainly applied in subgrade and subbase layers; however, they can also be used in base layers.

The laboratory and field results of a research effort, involving chemical stabilization of naturally wet and problematic clayey subgrade soils in south Louisiana, showed that cement-treated soil, followed by lime-treated soil provided the best-performance among subbase treatments evaluated [2]. Another research, carried out on lime stabilization of an expansive subgrade soil, showed that the application of lime provides better physical properties in terms of liquid limit, plastic limit and plasticity index; furthermore, it was revealed that lime increases the CBR value from 1%, for untreated soil, to about 30-50% by adding different percentages of lime. Also, the design of pavement thickness showed that adding the lime decreases the required thickness of the pavement for about 50-60% compared to the thickness required for a pavement over an untreated subgrade soil [3]. A research conducted on five construction sites, in Oklahoma, where the subgrade soils were stabilised with cement kiln dust and Class C fly ash, revealed improved resilient modulus values ranging from 7 to 46 times larger than those of the untreated soil [4].

There have also been some researches carried out to investigate the effect of a combination of cement and lime stabilization on improvement of subgrades. Results of a research on lime modified clay subgrades in Texas, with cement stabilization, revealed that using combinations of lime and cement increased the typical subgrade durability by 4.8 to 5.7 times greater than applying just lime after one year of exposure to in-place conditions [5]. A similar research on stabilizing expansive soils revealed that applying combined lime and cement additives shows some promise, with initial lime treatment improving the workability and the subsequent cement treatment improving the strength and resilient properties of the same subgrade [6].

The main objective of this study is to evaluate the behaviour of lime and cement treated base layers in unpaved roads. Such roads have stone aggregate layers, placed directly above soil subgrades, and they are mostly surfaced with sandy gravels for reasonable ride-ability; thus the granular layer serves as a base course and a wearing course at the same time [7]. Full scale laboratory simulations seem almost inappropriate for small projects because they are time consuming, labour-intensive and very costly. Therefore, a combination of laboratory experiments and computer simulation has been considered in recent years to diminish the full scale modelling disadvantages [8, 9]. This study comprises of two main parts: (1) the laboratory experiments; (2) the numerical simulation of different base courses over the same subgrade using finite element method to evaluate their performance under different loads. For numerical simulation, the strength parameters of the treated materials were estimated by application of estimating functions proposed by Sharma et al. [10].
EXPERIMENT PROGRAM

Laboratory investigations have been carried out to study some of the mechanical properties of the stabilized granular soils. Two different soil types, well-graded gravel and well-graded sand, with maximum particle size of 19 and 9 mm, respectively, have been stabilized with various amounts of lime and cement admixtures to form bounded materials with a variety of mechanical characteristics. Three samples were produced for each different design mixes and unconfined compressive tests were conducted on each sample after a curing time of seven weeks.

Materials

Aggregates

Well graded gravel and sand have been selected to be used as the base course. Their particle distribution boundaries proposed by ASTM, make them ideal. In fact, two different types of materials were selected in order to investigate the effect of maximum grain size in stabilization results and provide a vast band of mechanical characteristics; thus maximum particle sizes of 19 mm and 9 mm were selected for gravel and sand respectively. The particle distribution curves of applied materials are given in Figure 1.
Cement
Portland cement type II was used as the basic stabilizing agent to produce the main structural strength. Its resistance against mild acidic attacks has made this type of cement a favourable one in previous stabilizing projects, and ideal to this research.

Lime
In order to make the stabilized material more resistant to harmful acidic environmental effects and achieving more ductile behaviour, High Calcium Hydrated Lime \((\text{Ca(OH)}_2)\) was also mixed with the applied cement. This material is a more practical version of lime, due to its fine particle size, which makes the mixture procedure, and the chemical reactions, easier and less time consuming.

Mixture Types
In order to check the validity of the estimating functions a wide range of mechanical properties of stabilized materials were required. Therefore, five various design mixes for gravel materials and three for treated sand specimens were applied. Table 1 presents the properties of different designs. The amounts of optimum water content, \(w_{opt}\), used in sample production were obtained from the modified compaction tests according to ASTM D 698-78.
### Table 1: The properties of different mix designs

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>Cement Ratio (%)</th>
<th>Lime Ratio (%)</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>4</td>
<td>6</td>
<td>10.4</td>
</tr>
<tr>
<td>S-2</td>
<td>5</td>
<td>5</td>
<td>9.5</td>
</tr>
<tr>
<td>S-3</td>
<td>6</td>
<td>4</td>
<td>9.1</td>
</tr>
<tr>
<td>G-1</td>
<td>4.5</td>
<td>6.8</td>
<td>9.4</td>
</tr>
<tr>
<td>G-2</td>
<td>4.5</td>
<td>5.6</td>
<td>8.9</td>
</tr>
<tr>
<td>G-3</td>
<td>4.5</td>
<td>4.5</td>
<td>8.2</td>
</tr>
<tr>
<td>G-4</td>
<td>5.6</td>
<td>6.8</td>
<td>9.6</td>
</tr>
<tr>
<td>G-5</td>
<td>5.6</td>
<td>4.5</td>
<td>8.9</td>
</tr>
</tbody>
</table>

Various mechanical properties were obtained by altering the amount of cement and the lime/cement ratio for each soil type which is subsequently illustrated.

### Preparing Test Specimens

Three standard cylindrical samples (10 cm in diameter and 20 cm in height) were produced for unconfined compressive tests for each mix design. Samples were compacted in five layers by the maximum possible compaction energy to reach homogenous samples in elevation [11]. The amount of applied energy for each sample’s compaction was about 1124.2 N.m, which was attained by numerous compaction tests [12]. The compacted samples were submerged in water for seven weeks at room temperature before undergoing the unconfined compressive tests.

![Figure 2: Specimens’ curing at room temperature](image)

### Unconfined Compressive Tests

The unconfined compressive tests were carried out on cured samples according to ASTM C39-86. The curing time of seven weeks has been selected so as to drastically reduce any noticeable significant change in the samples’ strength. The unconfined compressive tests were conducted on each sample, from which the ultimate compressive strength, failure strain, and the unconfined elasticity modulus were extracted and collated in Table 2, below.
### Table 2: Summary of unconfined compressive tests’ results

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>Average Compressive Strength (kN/m²)</th>
<th>Average Ultimate Strain (mm/mm)</th>
<th>Average Modulus of Elasticity (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>1662</td>
<td>0.0095</td>
<td>174922</td>
</tr>
<tr>
<td>S-2</td>
<td>3110</td>
<td>0.0076</td>
<td>409175</td>
</tr>
<tr>
<td>S-3</td>
<td>5854</td>
<td>0.0079</td>
<td>699590</td>
</tr>
<tr>
<td>G-1</td>
<td>4163</td>
<td>0.0107</td>
<td>387485</td>
</tr>
<tr>
<td>G-2</td>
<td>4649</td>
<td>0.0135</td>
<td>342694</td>
</tr>
<tr>
<td>G-3</td>
<td>5395</td>
<td>0.0102</td>
<td>524628</td>
</tr>
<tr>
<td>G-4</td>
<td>5646</td>
<td>0.009</td>
<td>627434</td>
</tr>
<tr>
<td>G-5</td>
<td>6822</td>
<td>0.009</td>
<td>756274</td>
</tr>
</tbody>
</table>

### ESTIMATING RELATIONS

Different failure criteria have been presented for cemented and brittle materials [13-16]. In order to use the results of soil improvement in finite element simulations, the strength parameters of stabilized soil, cohesion and friction angle must be firstly determined by application of estimating relations. In a recent research it was concluded that the non-linear failure criterion presented by Sharma et al. has a good compatibility with the treated materials of this research [17]. The estimating relations of Sharma et al. [10] are described subsequently.

The non-linear failure criterion for weakly cemented sand, Eq. (1), is attained by means of triaxial shear tests:

\[
\tau = P_a (0.115 \frac{q_u}{P_a} + 1.242)(\frac{\sigma + 0.035q_u}{P_a})^n
\]

where \( \tau \) is the shear strength, \( q_u \), \( P_a \), and \( \sigma \) are differential axial pressure, atmospheric pressure and confining pressure (in kPa) respectively. \( n \) varies from 0.5 to 1.0 and must be calibrated for different soil types.

When the amount of \( \sigma \) is reduced to zero (unconfined compressive test), the amount of \( \tau \) equals to the amount of real cohesion of materials (\( C_r \)):

\[
C_r = P_a (0.115 \frac{q_u}{P_a} + 1.242)(\frac{0.035q_u}{P_a})^n
\]

And the tangent friction angle (\( \phi \)) at any arbitrary confining pressure of \( \sigma \) is calculated by following relation:

\[
\tan(\phi_t) = \frac{(0.115 \frac{q_u}{P_a} + 1.242)^n}{\left(\frac{\sigma + 0.035q_u}{P_a}\right)^{1-n}}
\]
In this research, the cohesion and friction angle of the mixtures are determined by the application of the estimating relations of Sharma et al [10].

FINITE ELEMENT SIMULATION

Model Geometry

Finite element simulations for each mix design were carried out using a commercial finite element analysis (FEA) package namely, PLAXIS 8.2. The information required for defining the soil characteristics to generate the models was obtained from a combination of laboratory experiments and estimating relations presented by Sharma et al [10]. Properties like elasticity modulus and dry density were extracted from laboratory tests. Strength parameters such as the cohesion and the internal friction angle were attained by implying the compressive strength in estimating relations (information presented in Table 2).

Plastic analysis on an axis symmetric soil body having by a 20 cm stabilized soil, with dimensions of 5 m and 3 m in height and diameter respectively, was performed with the special characteristics of each mix design. Figure 2 demonstrates the geometry of the model together with the applied meshing system. In order to obtain more precise results, the base layer and the upper 40 centimetres of the subgrade are set to have finer meshes.

Finally, a total of 8 finite element models were generated and run in more than 6 analysis phases for each model (at least 48 analyses totally). Also, an extra model was generated, using another type of subgrade under G-5, and run; the results of which are used in the discussion section.

Subgrade and Base Layer Geotechnical Properties

The modulus of elasticity (E) of the mixture types was obtained by the unconfined compressive test and has already been presented in section 2.4. In order to obtain the cohesion and friction angles of the mixture types, a combination of laboratory experiments and the relationship proposed by Sharma et al. (section 3) are applied. The calculation process and discussions are described in details in Azadegan et al [17]. Table 3 presents the amounts of cohesion and friction angles that are assigned to the subgrade and base materials in the finite element model.
Figure 2: Finite element analysis: model geometry (left), mesh generation (right)

Table 3: Estimated amounts of cohesions and friction angles

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>C (kPa)</th>
<th>φ (degrees)</th>
<th>n*</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1</td>
<td>240.47</td>
<td>36.95</td>
<td>0.5</td>
</tr>
<tr>
<td>S-2</td>
<td>502.67</td>
<td>41.68</td>
<td>0.5</td>
</tr>
<tr>
<td>S-3</td>
<td>1226.18</td>
<td>45.26</td>
<td>0.6</td>
</tr>
<tr>
<td>G-1</td>
<td>727.89</td>
<td>44.04</td>
<td>0.5</td>
</tr>
<tr>
<td>G-2</td>
<td>840.40</td>
<td>44.94</td>
<td>0.5</td>
</tr>
<tr>
<td>G-3</td>
<td>1090.23</td>
<td>44.17</td>
<td>0.6</td>
</tr>
<tr>
<td>G-4</td>
<td>1163.86</td>
<td>44.78</td>
<td>0.6</td>
</tr>
<tr>
<td>G-5</td>
<td>1531.83</td>
<td>47.33</td>
<td>0.6</td>
</tr>
</tbody>
</table>

* n is the applied power, in non-linear failure criterion, which led to more proper simulation results [17]

Also, the properties of the two soil types that were taken as the subgrade soil in the finite element model, namely soft clay and hard clay, are presented in Table 4.

Table 4: Subgrade soil properties applied in the finite element modelling

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>C (kN/m²)</th>
<th>φ (degree)</th>
<th>E (kN/m²)</th>
<th>Poisson's Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay</td>
<td>100</td>
<td>20</td>
<td>32000</td>
<td>0.25</td>
</tr>
<tr>
<td>Hard Clay</td>
<td>300</td>
<td>30</td>
<td>75000</td>
<td>0.20</td>
</tr>
</tbody>
</table>
Loading

The contact pressure at the interface of the tyre and the pavement is important for the determination of the structural response of the pavement. For flexible pavement design, commonly a circular contact area with the diameter that equals the tyre width is used [19]. In this research a 12.00R24 tyre type, which is one of the most commonly used tyre for straight trucks, was taken to calculate the contact pressure and the contact area. Normally, these tyres are produced with a width greater than 255 mm. In this research, the width of the tyre, which is also the diameter of the contact circular area, was taken as 300 mm.

Based on a Permanent International Association of Road Congress (PIARC) publication [20], the rear axle maximum allowable load of 3 axle straight trucks ranges between 8 and 28 tonnes in different countries of the world; however, in the majority of the countries, this axle load is 18 tonnes. Taking 18 tones as the axle load and considering the fact that the rear axle of a 3 axle straight truck is a dual tandem axle that consists of 8 tyres (8 circular contact areas), the contact pressure of one tyre is estimated to be approximately 320 kPa.

Aside from the mentioned contact pressure, a number of loads, namely 25, 50, 100, 200 and the collapse load (in kPa), are applied to the PLAXIS models to investigate the behaviour of the pavement systems under different contact pressures.

RESULTS AND DISCUSSION

This study has been carried out to investigate the effect of applying a combination of lime and cement in order to improve the mechanical characteristics of two untreated base courses over soft clay subgrades in unpaved roads. To this end, the two soil types were treated with various amounts of lime and cement to prepare the test specimens. Using the test results and utilizing the estimating relations recommended by Sharma et al. [10], the unknown mechanical characteristics such as cohesion and internal friction angel were obtained and applied in the numerical simulations. The results of the analysis are discussed from two aspects: vertical deformation under a specific load, and the collapse load.

Vertical Deformation under a Specific Load

In this research, the specific load that was taken to be applied on the models is 320 kPa, as discussed in section 3.2. Figure 4 illustrates the maximum vertical deformation of the 8 base layers at the center of the load contact area.

The mixture types are sorted from left to right based on the increment of the modulus of elasticity. As it was predicted, the increment in the modulus of elasticity of the mixture results in lower deformation under the same load.

The Collapse Load

Figure 5 illustrates the maximum applicable load that can be applied to each model before it collapses, simply mentioned as the collapse load, together with the corresponding modulus of elasticity.
Figure 4: Maximum vertical deformation of the mixture types under a contact pressure of 320kPa (Sorted with the increment of E from left to right)

Figure 5: Collapse load and the modulus of elasticity of the mixture types.

Surprisingly, G-5 and S-3 (the two stiffest mixtures) that have the lowest deformation under a specific load, presented the lowest collapse load. In other words, these mixtures present the lowest bearing capacity compared to the others. Table 5 elaborates on this matter, where, columns 2 and 4 from left, X (m), present the horizontal distance from the centerline of the load where the maximum shear stresses appear in horizontal cross sections at the depths of 10 cm (Section A-A) and 20 cm (Section B-B) (Fig. 6). The ratio of the maximum shear stress to the collapse load of the mixture types, Rs (%), at the depth of 10 cm and 20 cm (the interface of the base layer and subgrade) are also listed in Table 5 (columns 3 and 5 respectively). Finally, the last column represents the percentage of reduction in Rs from section A-A to section B-B.

As it is observed, the highest reduction belongs to the stiffest base layer, i.e. G-5, and the lowest reduction belongs to S-1, which has the lowest stiffness. It can be concluded that in the S-1 base layer the stress distribution is performed better in depth rather than in the G-5 base layer.
Table 5: Values of Rs at the depth of 10 cm and 20 cm, together with percentage of reduction in Rs (%)

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>Section A-A Y= -0.10 m</th>
<th>Section B-B Y= -0.20 m</th>
<th>Reduction in Rs (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X (m)</td>
<td>Rs (%)</td>
<td>X (m)</td>
</tr>
<tr>
<td>S-1</td>
<td>0.125</td>
<td>28.4</td>
<td>0.191</td>
</tr>
<tr>
<td>S-2</td>
<td>0.163</td>
<td>31.0</td>
<td>0.193</td>
</tr>
<tr>
<td>S-3</td>
<td>0.166</td>
<td>30.7</td>
<td>0.193</td>
</tr>
<tr>
<td>G-1</td>
<td>0.147</td>
<td>31.2</td>
<td>0.194</td>
</tr>
<tr>
<td>G-2</td>
<td>0.168</td>
<td>28.1</td>
<td>0.19</td>
</tr>
<tr>
<td>G-3</td>
<td>0.145</td>
<td>30.2</td>
<td>0.194</td>
</tr>
<tr>
<td>G-4</td>
<td>0.168</td>
<td>29.3</td>
<td>0.193</td>
</tr>
<tr>
<td>G-5</td>
<td>0.168</td>
<td>30.0</td>
<td>0.192</td>
</tr>
</tbody>
</table>

Figure 6: Schematic view of cross sections to investigate shear stresses

The stress distribution in depth of the subgrade is highly dependent on the difference between the stiffness of the base layer and the subgrade. If the stiffness ratio of base/subgrade is high, a great proportion of the energy will be absorbed by the base layer and the contact pressure will be concentrated in a limited zone under the tyre and base layer interface instead of being distributed in the depth of it. This limited stress zone of a stiff pavement layer can lead to a premature destruction due to the fatigue stress. As an example, in spite of the fact that G-5 has the lowest deformation under the load of 320 kN/m² and possesses the greatest E among the other mixtures, it presented the lowest bearing capacity. In order to prove the aforementioned claim, the existing subgrade soil was replaced with a stiffer one, i.e. hard clay (Table 4) to generate a new model. The analysis revealed that the collapse load of the mixture increased from 392.5 kPa to 900 kPa. This is due to a lower stiffness ratio of base/subgrade, or in other words, a higher correspondence between the base layer and the stiffer subgrade.

It should also be noted that a reduction in the stiffness leads to greater static deformations that appear in long time, such as the rutting distress. For instance, in spite of the fact that G-1 and S-2
showed a higher collapse load compared to S-3 (Figure 5), the latter showed a lower vertical deformation under a specific load (Figure 4). Figure 7 shows the increased percentage of deformation when using each of the mixture types for the base layer, rather than G-5, under the load of 320 kN/m² in the depth of the model. Since G-5 evolves the greatest modulus of elasticity among the mixtures, all the percentages are greater than zero, i.e. the deformation of the pavement system increases by the reduction of the stiffness of the base layer.

Figure 7: Increased percentage of deformation of the mixture types compared to G-5 under the load of 320 kN/m².

Finally, based on the discussions in the previous paragraphs and Figure 6, G-2 and S-1 seem to be the most susceptible mixtures for rutting distress.

CONCLUSIONS

In this research, the performance of 8 types of lime and cement treated base layers over soft clay subgrade soils was investigated. To this end, two different granular soils, well graded gravel and well graded sand, were treated with various amounts of lime and cement to produce compressive test specimens (8 mixture types). Since performing comprehensive laboratory experiments on stabilized soils to get all parameters required for finite element simulations are very time consuming and expensive, hence almost impossible for small projects, the required characteristics of stabilized soils were estimated by performing unconfined compressive tests and applying the non-linear failure criterion proposed by Sharma et al. The estimated material properties were then assigned in the PLAXIS 8.2 models to compare the vertical deformations of the 8 base layer types under a specific load, and to determine the maximum bearing capacity of each.

Based on the results of the numerical analysis, the following points are recommended for choosing the most proper mixture type to construct a base layer:
Aside from the collapse load of the base layer, the correspondence of the base and the subgrade should also be noted, since a great difference in stiffness (between the two layers) can cause a great reduction in the bearing capacity of the base layer.

Aside from the collapse load of the base layer, the stiffness of the base layer should be noted, since lower stiffness can result in greater deformations, which under prolonged conditions can cause rutting distress.

REFERENCES


