The relationship between California Bearing Ratio and elastic stiffness for compacted clays
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Introduction

For pavement design purposes, subgrades are generally characterised by the California Bearing Ratio (CBR) (see Croney, 1977). This is a penetration test developed for use with empirical pavement design methods. It is not directly applicable to the analytical approach to design (Brown et al., 1984). However, since the CBR test is in widespread use, correlations with elastic stiffness have been proposed and are used in design computations in the absence of more reliable data. The proposed correlations between resilient Young's modulus ($E_r$) and CBR include:

$$E_r = 10 \text{ CBR MPa}$$

based on widespread field vibration testing of soil and granular materials (Heukelom & Klomp, 1962) and

$$E_r = 17.6 \text{ CBR}^{0.64} \text{ MPa}$$

developed by the Transport & Road Research Laboratory (Powell et al., 1984). In each case the CBR is expressed as a percentage.

The problems inherent in attempting to correlate soil stiffness with CBR were discussed by Hight and Stevens (1982). The principal objection to the CBR test for use in the determination of basic soil properties is that the effective stress state of the test specimen is unknown. The nature of the test is such that a wide range of stress conditions is induced in the soil. Close to the load plunger, shear failure develops while, further away, only modest stresses are developed. The interpretation of the data to determine the CBR value involves use of the loads corresponding to 2.5mm and 5mm penetration of the plunger. These do not correspond to complete failure but they do represent conditions of very high strain in the neighbourhood of the plunger. Furthermore, the test is monotonic. Consequently the CBR value is likely to be influenced both by the shear strength of the soil (for a clay in the undrained situation) and by its monotonic stiffness.

For pavement design purposes, an elastic or resilient stiffness is required, based on repeated or cyclic loading. The elastic response of the subgrade to wheel loading influences the stresses and elastic strains induced in the upper layers of the pavement and they, in turn, determine the likelihood for failure to develop through cracking or rutting.

Current methods involve use of the vertical elastic strain at the top of the subgrade as a semi-empirical general indicator of the likelihood

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<table>
<thead>
<tr>
<th>Property</th>
<th>Keuper Marl</th>
<th>Gault Clay</th>
<th>London Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.69</td>
<td>2.69</td>
<td>2.73</td>
</tr>
<tr>
<td>% Clay</td>
<td>33</td>
<td>39</td>
<td>54</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>18</td>
<td>25</td>
<td>23</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>37</td>
<td>61</td>
<td>71</td>
</tr>
<tr>
<td>Plasticity Index (%)</td>
<td>19</td>
<td>36</td>
<td>48</td>
</tr>
</tbody>
</table>

Table 1: Basic material properties of the soils.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Sample No</th>
<th>Water Content (%)</th>
<th>Liquidity Index (%)</th>
<th>Suction (kPa)</th>
<th>Density (kg/m³)</th>
<th>Degree of saturation (%)</th>
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</thead>
<tbody>
<tr>
<td>Keuper</td>
<td>1</td>
<td>24.5</td>
<td>34.2</td>
<td>25.5</td>
<td>1530</td>
<td>88.0</td>
</tr>
<tr>
<td>Marl</td>
<td>2</td>
<td>21.4</td>
<td>17.9</td>
<td>48.0</td>
<td>1640</td>
<td>90.7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>18.7</td>
<td>3.7</td>
<td>72.0</td>
<td>1745</td>
<td>92.6</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>16.6</td>
<td>-7.4</td>
<td>93.5</td>
<td>1820</td>
<td>93.7</td>
</tr>
<tr>
<td>Gault</td>
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<td>39.0</td>
<td>38.0</td>
<td>18.0</td>
<td>1230</td>
<td>88.7</td>
</tr>
<tr>
<td>Clay</td>
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<td>31.9</td>
<td>19.2</td>
<td>37.5</td>
<td>1390</td>
<td>92.1</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>27.1</td>
<td>5.8</td>
<td>56.0</td>
<td>1525</td>
<td>95.4</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>22.5</td>
<td>-6.9</td>
<td>77.5</td>
<td>1670</td>
<td>98.6</td>
</tr>
<tr>
<td>London</td>
<td>1</td>
<td>40.3</td>
<td>36.0</td>
<td>19.0</td>
<td>1245</td>
<td>92.2</td>
</tr>
<tr>
<td>Clay</td>
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<td>18.3</td>
<td>43.0</td>
<td>1410</td>
<td>92.7</td>
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<tr>
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<td>76.0</td>
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<td>24.4</td>
<td>32.5</td>
<td>1330</td>
<td>90.0</td>
</tr>
</tbody>
</table>

Table 2: Details of triaxial test specimens.

for rutting to develop within a given number of load applications (Brown and Brunton, 1984). The computation of this parameter also relies on a knowledge of the elastic stiffness of the soil. A further complicating factor in pavement analysis is that both clay soils and granular materials exhibit markedly non-linear elastic properties (Brown et al., 1975, Pappin and Brown, 1980). Consequently, a unique value of elastic stiffness for the subgrade does not exist. It is common practice, however, at least for heavy duty pavements, to assume a single value of stiffness for the soil in design computations, since the structural response of the system is dominated by the stiffness characteristics of the asphalt or concrete layer and only certain key parameters are required for design at specific locations (Brown et al., 1984).

In computations where the detailed deformation behaviour of the subgrade becomes important, its non-linear response must be properly modelled. Such cases include back-analysis of surface deflection measurements to deduce layer stiffness in structural evaluation procedures (Brown et al., 1986). In such circumstances, a single value of stiffness correlated with CBR is clearly inadequate.

The data reported in this paper highlight the non-linear response of three compacted clays and the relationship between resilient stiffness and CBR is examined.

The soils tested

Three soils were tested in the experimental programme; Keuper Marl (more properly known as Mercia Mudstone), Gault Clay and London Clay. Their basic properties are given in Table 1. Before use, each soil was dried and powdered. Samples were prepared by adding distilled water to the powder until the required water content was attained. The moist soil powder was thoroughly worked using a pallette knife to avoid regions of dry soil. Details of the compaction procedures for each type of test are given below.
Figure 1: Results from suction-water content tests using the rapid suction method.

The experiments

Independent series of repeated load triaxial tests and CBR tests were performed. The results of the triaxial tests were related to those of the CBR test by means of the soil moisture contents which were, for each soil type, assumed to be uniquely related to the soil suction. The relationship between water content and suction for each soil was determined in an ancillary series of tests using the Rapid Suction Apparatus developed by the TRRL (Dumbleton and West, 1968). The soil was mixed at high moisture content and ‘dried down’ in stages by the addition of soil powder with suction measurements being made at the end of each stage. The results are summarised in Figure 1 showing values obtained as the water content reduced.

Triaxial testing

The water contents, densities and related data for all the triaxial specimens are given in Table 2. The soil suction values were determined from the water contents using the curve in Figure 1. The range of water contents selected for the Keuper Marl specimens represented the likely in-situ conditions and was based on related work on saturated material described by Loach (1987). The water contents used for Gault Clay and London Clay were such that a broadly similar range of soil suction was obtained for each soil. This was considered desirable as both CBR and elastic stiffness have been shown to depend on the level of suction (Black, 1962, Brown, 1979). For the range considered, a reasonably unique relationship was demonstrated between soil suction and liquidity index as shown by Figure 2. This is useful for estimating soil suction and, hence, both CBR and elastic stiffness as illustrated by the results which follow as well as the previous work referenced above.

When thoroughly mixed at the correct water content, the soil was compacted into a cylindrical split mould 75mm in diameter and 150mm long using hand tamping with a 12mm diameter rod. Following compaction, the sample was removed from the mould and fitted with a neoprene membrane. On-sample instrumentation was then fitted to allow axial deformation to be determined using four LVDT’s measuring over a central gauge length using the principle shown in Figure 3. Radial deformations were measured with a strain gauged hoop (described by Boyce and Brown, 1976) but the results proved unreliable and were not used. Following manufacture, the specimens were allowed to stand for one day to improve the water content uniformity and were then tested unconfined.

The deviator stress was applied using a pneumatically controlled actuator operating at a constant frequency of 1Hz, the deviator load being measured by a strain-gauged load cell immediately above the sample. The magnitude of the deviator stress pulse was increased in stages. After 100 cycles at each level the magnitude was increased until axial resilient strains of about 2500 microstrain were measured. There was some reduction in stiffness between cycles 1 and 100 at the higher stress levels but this was never more than about 10%.

Table 3: Experimentally-derived material constants for elastic stiffness and CBR.

Consequently, the axial strains at cycle 1 were used to develop the relationships that follow and these are limited to strains up to 1000 microstrain so that the reduction in stiffness effect noted above was eliminated. The corresponding deviator stress pulse magnitude are below 50kPa, corresponding to the range likely to be experienced in a pavement subgrade (Loach, 1987).

Plots of axial resilient strain against deviator stress pulse were constructed for the various soil types tested, and the levels of deviator stress corresponding to axial resilient strains from 200 to 1000 microstrain at 200 microstrain intervals were read off. These results are presented in Figure 4, in the form of contours of resilient axial strain ($e_{ax}$) for each of the three soils using soil suction as the independent variable. Samples number 4 for Keuper Marl and Gault Clay exhibited much stiffer behaviour than the others. This may be related to the fact that the samples were below the plastic limit and, therefore, had negative liquidity indices (see Table 2).
The equations of the contours fitted to the data points in Figure 4 may be expressed as:

\[ \varepsilon_{\text{ref}} = A \left( \frac{q}{S} \right)^B \]  

where A and B are material constants, \( q \) is the deviator stress pulse and \( S \) is the suction. The coefficients A and B are given in Table 3. In terms of resilient Young's modulus \( (E_r) \), equation (3) leads to:

\[ E_r = \frac{q}{A} \left( \frac{S}{q} \right)^B \]

California Bearing Ratio tests

Each specimen was prepared by compaction in a CBR mould using the same procedure as for triaxial specimens and was then allowed to stand for one day. Monotonic penetration testing was performed in accordance with the procedure set out in BS1377: 1975. The resulting CBR values were plotted against suctions obtained via water contents using Figure 1. The data points, shown in Figure 5, were fitted to a series of simple relationships given by:

\[ \log \text{CBR} = C \cdot S + D \]

The values of the material constants C and D are given in Table 3.

The relationship between stiffness and CBR

Since both resilient Youngs modulus \( (E_r) \) and CBR are related to the suction, it is possible to derive a relationship between \( E_r \) and CBR. Combining equations (4) and (5) gives:

\[ E_r = \frac{q}{A} \left[ \frac{\log \text{CBR} - D}{C \cdot q} \right]^B \times 10^{-3} \]  

where \( E_r \) is in MPa and \( q \) is in kPa. Figure 6 shows how the resilient Young’s modulus for Keuper Marl is related to the CBR using the model described by equation 6. As can be seen, the magnitude of the repeated deviator stress pulse has a significant effect. When the repeated deviator stress has a magnitude of 60 kPa, the empirical predictions of \( E_r = 10 \times \text{CBR} \) and 17.6 CBR\(^{0.64}\) are virtually coincident with the prediction of equation (6) for CBRs in the range 5%-10%.

Figure 7 shows the relationship between the predicted resilient Young’s modulus of all three soils and CBR for a stress pulse amplitude of 40 kPa, which is considered to be a typical in-service value for the top of the subgrade. As can be seen, the Marl is much stiffer than either Gault or London Clay for any given value of CBR. The empirical relations are again shown for comparison and appear to provide only a rough guide to the expected soil stiffness, although the TRL equation is of the correct shape.

The trends in Figures 6 and 7 show that, if accurate predictions of soil stiffness are to be obtained, more sophisticated models are required than those provided by equations (1) and (2) above. In particular, the soil type and stress levels must be taken into account. With respect to the soil type, the main disparity in the present series of tests is between the stiffness of the Keuper Marl (which is a rather silty clay with a plasticity index of 19%) and the other two soils, which have plasticity indices between 36% and 48%. Hence, it may be possible to use plasticity index as a normalising factor in a new model. It is considered, however, that at this stage insufficient data exists to establish the validity of this approach or the form of any possible relationship.

Conclusions

Experiments with remoulded samples of three commonly occurring British clays, tested under conditions appropriate to pavement subgrades, demonstrated:

1. Resilient Young’s Modulus depends on the soil suction, the magnitude of applied deviator stress pulse and the soil type.
2. There is not a unique relationship between resilient Young’s Modulus and CBR because of the influence of deviator stress.
3. For soils compacted at water contents greater than the plastic limit the relationship between resilient Young’s Modulus and CBR for a given deviator stress pulse magnitude is of the same type as...
that proposed by TRRL, viz, \( E_r = 17.6 \times CBR^{0.64} \) (MPa) but this equation does not take account of soil type. It is, however, an improvement on the simpler relationship, \( E_r = 10 \times CBR \) (MPa) which has been extensively used in the past.

Acknowledgements

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