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The effects of subgrade clay condition on the structural behaviour of road pavements

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ABSTRACT: This study examines deflections of a pavement under construction and the effects on it of the deformability of clays for different soil-water conditions. This analysis is based on a simple layered elastic analysis using a stress-strain model applicable for saturated and unsaturated conditions, established from the results of repeated load triaxial testing. Analysis of data from repeated load triaxial tests carried out on recompacted soils has enabled a new model to be developed. A simple plasticity-related version is presented for routine use.

1. INTRODUCTION

The deformation of soils in pavement structures is important as the constructibility and long-term performance depend on the magnitude of both resilient and permanent deformations being kept small. Calculation of the level of defomations is always difficult as it requires:

(i) a knowledge of the way in which soil behaves when subjected to the complexities of pavement loading.

(ii) a quantification of a particular material's chatacteristics at the level of stress and pore water pressure relevant to the design circumstances.

Calculation is made much more complex in that many soils beneath pavements are recompacted and desaturated, either accidentally or deliberately, by the construction process.

This paper investigates the effects of changing the saturated state of some clays on the deformation behaviour of the pavements at the construction stage, when the effect of the clay's varying condition is most significant for performance.

2. THE DESIGN PROCESS

2.1. *Design Criteria for Soils in Flexible Pavements*

The soil layers of a pavement have to carry the stresses passed down either by direct trafficking or through higher layers. The stresses imposed are progressively lessened as construction proceeds. Most intense are the contact stresses imposed by the earthworks plant. However, this case is not usually considered in design, as deformed soil can be removed by the plant immediately prior to placement of the overlying layer (usually a granular material).

The first, and most critical, design case for practical purposes is when the aggregate layer has been placed and is carrying construction traffic. Many passes, perhaps as many as 1000 (depending on the length of road under construcrion (Hardman et al., 1976)), of heavy vehicles must be carried by the partially completed pavement. There are two criteria which are required to measure success:

i) Surface rutting of the aggregate layer. High surface ruts can be repaired, but they are usually associated with rutting at the aggregate-soil interface, which cannot. The permissible surface rutting is commonly set at 40mm (Powell et al.,1984).

ii) Resilient deformation at the pavement surface. If this is too high the paving plant will not be able to satisfactorily compact the bituminous layers of the pavement. Low densities in, or roller-induced cracking of, the bituminous layers may result. Excess resilient surface deformation is due to inadequate stiffness of either or both aggregate and soil layers.

The second design case is for the completed flexible pavement subjected to a much larger number of load passes by conventional traffic and, once again, two criteria are applied. This time they are:

i) The asphalt tensile strain at the bottom of the asphalt layers, which is a function of the thicknesses and stiffnesses of the constituent pavement layers.

ii) The vertical subgrade strain at the top of the soil layers. This is used as a semi-empirical criteria which is related to the propensity for the whole pavement to rut for a given bituminous material

(Brown & Brunton, 1984). As the stress levels passed down to the soil and aggregate layers are small compared to the construction case described above (which must already have been succesfully completed) rutting due to soil deformation under normal trafficking is rarely a problem.

The sensitivity of the completed pavement to soil condition was outlined in a paper by Brown & Dawson (1987) when a saturated soil's stiffness was considered as a function of the position of the water table.

This paper is concerned with the sensitivity of the resilient deformations of the partially completed pavement to the subgrade soil condition. In particular this is considered with the intention of improving the design process. Estimated deformations will depend on both the soil condition and on the model used to describe the soil's stiffness at this condition. Both of these factors are considered. In particular the effect of having non-saturated, remoulded, soil in the pavement structure is reviewed.

2.2. *Design Computational Techniques*

Sophisticated computer based analysis techniques, usually based on a finite-element approach are available to assist with design. However, these methods are seldom "user-friendly" and are rarely written in such a way that resilient geotechnical constitutive relationships can be easily incorporated. Hence the pavement engineer wishing to carry out a design study normally relies on a layered elastic computational method. These have several advantages for all but the most detailed study:

1) easy to use;
2) well attested by frequent use;
3) convenient (usually will operate on a personal computer);
4) quick.

In the study reported here the layered elastic program "ELSYM 5" was used (Federal Highways Administration, (1985)).

3. PROBLEMS OF QUANTIFYING SOIL CONDITION

Brown and Dawson (1985) illustrated the typical stress history for an element of undisturbed subgrade (Fig.1). Soil will arrive at the conditions indicated by point A as a result of overconsolidation. This overconsolidation may, (given time), be increased or decreased due to unloading by the earthworks operation. The stress path followed by excavated and recompacted soil which had been at condition A is complex. In general it will end up at a value of q higher than A and somewhere beneath the failure line. Such soil is unlikely to be heterogeneous; some parts having been taken to failure, others not. In the short-term it is likely to be comprised of saturated clods with air-filled voids between. In the long term a degree of equilibrium between air and water in each void will probably be reached.

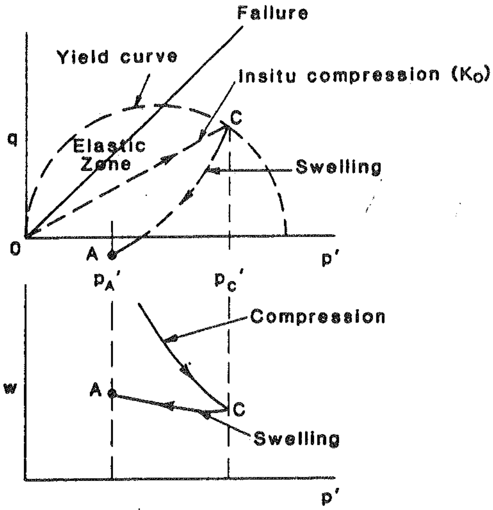

Figure 1 Typical Subgrade Stress History (after Brown & Dawson, 1985)

The positive or negative pore pressure (suction when external load is zero) resulting from recompaction is also in doubt. In the short-term the shearing due to compaction is likely to generate noticeable suctions in the saturated parts whilst the inclusions of air are likely to induce tensions in the unsaturated part. How long it takes for these to equilibrate with each other is not known. In the longer term the pore pressure condition will be controlled by the externally imposed water-table ré gime following appropriate drainage or swelling.

These uncertainties of soil condition and degree of equilibration force designers to make conservative assumptions about the suction level in soil. In the study reported here, the suction was computed as the long-term value in equilibrium with the water table (Black and Lister, 1976), whilst the mechanical properties were based on parameters obtained from repeated-load triaxal testing of recompacted clay samples which had been allowed to come to a moisture equilibrium with an externally applied suction level.

4. SOME BEHAVIOURAL MODELS FOR CLAY SUBGRADE MATERIALS

Loach (1987) carried out repeated load triaxial testing of an anisotropically overconsolidated saturated marl and obtained an equation relating resilient modulus, M_r (defined as the repeated axial stress divided by the resilient axial strain), to stress conditions:

$$M_r = A\left(\frac{p'_o}{q_r}\right)^B q_r \qquad (1)$$

where: p'_o is the mean normal effective stress before repeated loading.

q_r is the repeated deviator stress.

A, B are material constants, with B=1.5. (Brown & Dawson, 1987)

A similar model was also put forward by Loach (Brown et al., 1990) for a similar but compacted and unsaturated marl and two other soils.

$$M_r = A\left(\frac{S}{q_r}\right)^B q_r \qquad (2)$$

where S is the soil suction. On this occasion B had a value of 2.2.

For a similarly unsaturated compacted Kaolin clay (w_p = 32, w_L = 52), Gomes Correia (1985) suggested:

$$M_r = A\left(\frac{p'_o}{q_r}\right)^B \qquad (3)$$

where B was a little greater than unity. In his tests, Gomes Correia was able to apply a controlled suction and hence p'_o and S are synonymous in his experiments.

Both of these last two models were developed from repeated load, unconfined, triaxial test data.

Determination of the parameters A and B is complex in each case, requiring sophisticated laboratory equipment, and a simplification was therefore sought. Firstly Loach's data (Loach, 1987) for recompacted specimens was replotted (see, as an example, Fig.2). Initially, it was noted that Loach's model, used with the parameters he had deduced, failed to reproduce his own data at low deviatoric stress levels. It was also observed that stiffness was an inverse linear function of q_r for a particular level of suction. This tends to confirm the near unity power low proposed by Gomes Correia (1985) (see Eqn. 3). Several reaearchers have observed that resilient deformation is related to the proximity to failure attained by the stress path traversed by the tested soil. This is reflected in the ratio p'_o/q_r in the equations above.

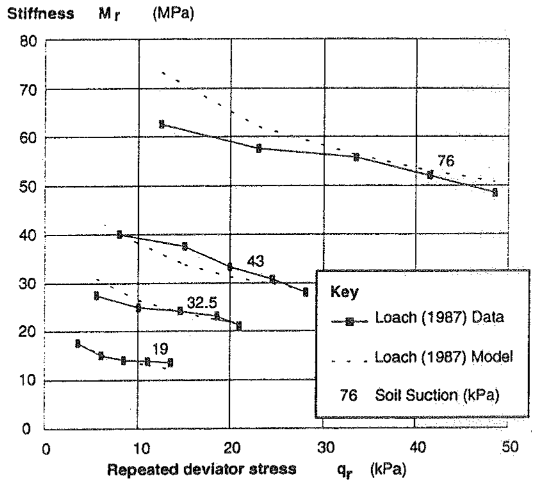

Figure 2 Resilient behaviour of recompacted unconfined samples - Loach (1987) model

Figure 2 also indicates that an equation relating M_r to q_r will have a finite maximum value at $q_r = 0$ Hence an equation of the form

$$M_r = f_n\left(A - B\frac{q_r}{p'_o}\right) \qquad (4)$$

for a level of suction is suggested. Higher suction levels evidently give higher stiffness. To model this aspect the right hand side of Equation 4 could be multiplied by the suction or initial effective stress, p'_o). No test results are available at zero suction but, it is assumed, the stiffness would not be precisely zero in such a condition. Thus a third parameter, C, is introduced for such an eventuality. Thus, the general equation

$$M_r = C + Ap'_o - Bq_r \qquad (5)$$

is proposed. Figure 3 shows that such an equation does indeed give a reasonable fit to the available data for recompacted unsaturated materials and is probably as accurate as the more sophisticated models discussed earlier. Whilst the predictions at low deviatoric stress levels are likely to be significantly improved, it is evident that:

i). The relationship to suction level is somewhat oversimplified by the new approuch.

ii). At high suction levels the sensitivity of stiffness to deviatoric stress increases.

This latter point may reflect the effect of increasingly low saturations making volumetric strains (rather than shear strains) of more significance or may reflect instrumentation inaccuracies at the very low strain levels concerned. For each material plotted, separate values of constants A, B and C are

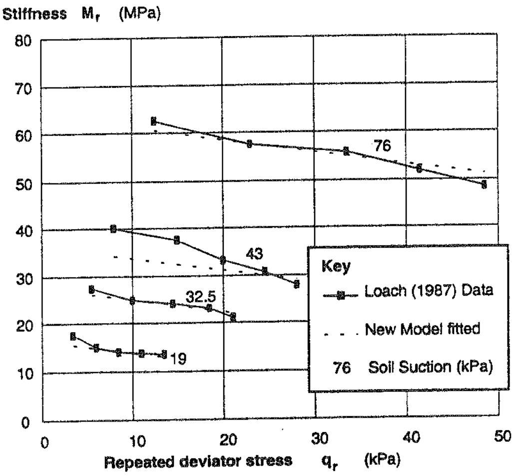

Figure 3 Resilient behaviour of recompacted unconfined samples - New approach

used. These are tabulated in Table 1. The equations so formed apply for values of stiffness which may be expected to exist beneath a pavement.

Equation 5 is very similar to that proposed by Chaddock (1982) who presented the relationship (in kPa)

$$M_r = 116000 - 90p'_o - 1150q_r \quad (6)$$

for an undisturbed natural Sandleheath clay ($w_P = 24$, $w_L = 68$), although the negative constant for p'_o is anomalous indicating reducing stiffness at higher ambient stress levels which was not observed by Loach (1987) or Gomes Correia (1985) or, indeed, by those other researchers using a q_r/p'_o formulation.

Table 1 Material Constants for Eqn. 5, as used in Fig. 3, together with Reference Soil data.

Material Constant	C (MPa)	A	B	w_P	w_L	Tested at Suction (kPa)
Keuper Marl *	-16.12	1.98	1.35	18	37	26 - 94
Gault Clay *	-6.9	0.95	0.64	25	61	18 - 78
London Clay *	0.67	0.83	0.26	23	71	19 - 76
Kaolin Clay **	10.42	1.03	-0.52	32	52	15 - 60

* Loach (1987)

** Gomes Correia (1985)

Equation 5 is a simple model but would still require sophisticated testing in order to determine the material parameters A, B & C. Accordingly a normalizing parameter for different soils was sought. Of those tried Plastic Limit (w_P) appeared to give the best results. All the available data, except for a few results at very high stiffnesses (100-180 MPa), for reasons mentioned above, are plotted on Fig. 4.

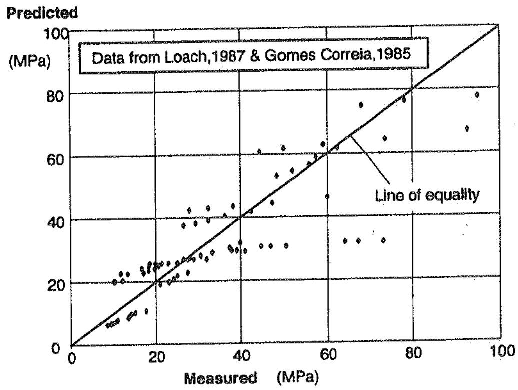

Figure 4 Resilient moduli estimation using generalised model

The predicted stiffnesses came from the equation:

$$M_r = 49\,200 + 950\,p'_o - 370q_r - 2\,400\,w_P \quad (7)$$

for convenience M_r, p'_o and q_r are in units of kPa and w_P as a percentage. This equation is applicable to soils which have their suctions and moisture content in equilibrium and when M_r values less than or equal to 80 MPa are predicted.

Measurement of Plastic Limit and calculation of imposed stress levels is thus sufficient to make a reasonably accurate assessment of the stiffness of these recompacted clays.

It is interesting to note that Equations 5, 6, and 7 all indicate a finite maximum stiffness as unstressed conditions are approached. The maximum stiffnesses revealed by the data discussed here is less than 200 MPa and the accuracy of these high values is in some doubt as already mentioned. Stiffness at low values of shear strain is important for earthquake loading of soils. Two results of particular relevance to the current study are those of Hicher et al. (1987) who determined a maximum value of M_r of less than 250 MPa for an undisturbed marl at a p'_o of 96 kPa and a maximum value of 230 MPa for a remoulded marl at a p'_o of 200 kPa and, for sand, those of Shen et al. (1985) who obtained a maximum resilient modulus of 100 MPa at a confining stress, p'_o, of 42 kPa. Data from Hicher et al. (1987) has been replotted in the form used in this paper (Fig.5). It will be seen that the form is the same as that used here. The maximum stiffness is clearly visible. (The slightly irregular line is due to the abstraction of the data from Hicher's tiny figure).

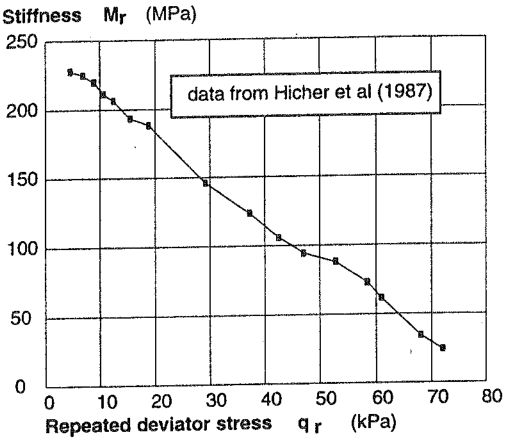

Figure 5 Resilient behaviour of remoulded marl

An alternative method of obtaining resilient modulus is to use an empirical correlation of M_r to CBR. Such a method for recompacted soils under an equilibrium suction was proposed by Black and Lister (1979) and developed by Blood and Lord (1987). Using this method the imposed suction level can predict the equilibrium moisture content from a wetting or drying curve, normalized with respect to the soil's plasticity. This moisture content is then empirically related to CBR for a particular plasticity and the CBR to modulus on the basis of the empirical relationship:

$$M_r = 17.6\ \mathrm{CBR}^{0.64}\ \text{(MPa)} \qquad (8)$$

due to Powell et al. (1984). In this approach there are thus several imprecise or empirical relationships between defining suction and obtaining stiffness.

5. PAVEMENT FOUNDATION ANALYSES

To illustrate the effect of material condition on the deformation of the pavement, and to see whether or

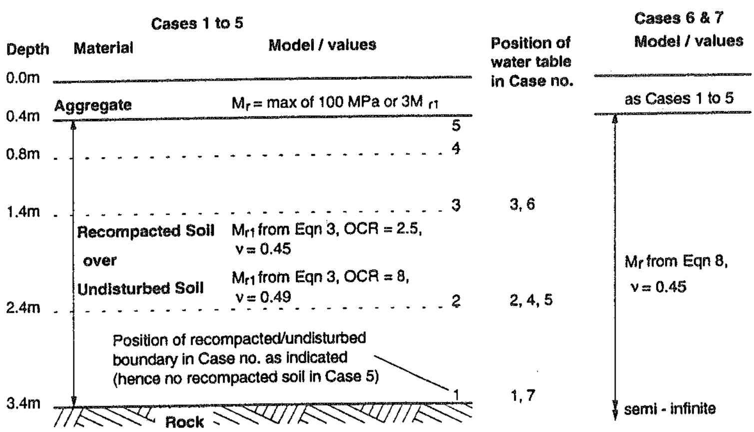

Figure 6 Pavements Analysed

Table 2 Series of Pavements Analysed and Results.

Case		1	2	3	4	5	6	7
Water table at depth of (m)		3.4	2.4	1.4	2.4	2.4	1.4	3.4
Undisturbed soil at depth of (m)		3.4	2.4	1.4	0.8	0.4	-	-
Computed Stiffness (MPa)	- in aggregate	60	50	30	40	70	7	100
	- top soil layer	20	15	9	15	22	27	31
	- second soil layer	53	41	27	58	60	-	-
Vertical Displacements (mm)	- at surface	2.9	4.2	6.9	4.9	3.0	3.6	2.5
	- at 0.4 m	1.5	2.2	3.5	2.3	1.6	2.1	1.5
	- at 0.6 m	0.9	1.2	1.9	1.2	0.9	1.6	1.1

not non-linear soil models can give realistic results using conventional layered elastic computational techniques a series of pavemants were analysed. Each comprised 400mm of granular material overlying clay (Fig.6). The water table and depth of recompaction varied; details are given in Table 2. The assumed properties are given on Figure 6.

The aggregate was assumed to have a maximum stiffness of 100 MPa, often reduced to three times the upper clay stiffness to reflect compaction difficulty on weak soils. The recompacted soil's stiffness was computed, for each layer, using Equation 3; p'_o being based on an overconsolidation ratio (OCR) of 2.5 (after Loach,1987). For the particular recompacted clay considered, values of material constants for Equation 3 were taken from Gomes Correia (1985). On the same evidence a maximum stiffness of 100 MPa was used. For the two empirical designs (numbers 6 & 7), based on the method summarized at the end of the last section, equilibrium suction values were determined using the equations:

$$S = u - \alpha p \tag{9}$$

$$\alpha = 0.0231\ I_P + 0.007 \tag{10}$$

Equation 10 is due to Verbrugge (1972).

The same approach was used for the saturated virgin soil (the lower layers in Cases 2-5), except that an OCR of 8 was chosen (after Loach, 1987). For the last two cases, a two layer approach was used. Estimated stiffnesses were provided for cases 1 to 5, q_r computed at several radial points, a mean value over the zone of significant stressing taken and the pavement reanalysed using a new value of M_r based on the new q_r value. This was continued until the solution had converged; a maximum of 4 computation cycles. In all the analyses a 65 kN load with a 500 kPa tyre pressure was assumed.

The results are given in Table 2. Computed values of q_r varied little for all the cases examined. The resilient modulus values of material beneath about 2m were always at the limiting value of 100 MPa. The strain measurements rank the performance in the same way as the displacements do with the exception of cases 6 and 7. There, the semi-infinite layer at a middle-range stiffness results in higher strains being computed.

6. DISCUSSION OF RESULTS

From Table 2 it will be immediately obvious that a higher water table and greater depth of recompacted clay (Cases 1, 2 and 3) results in a large increase in deflections at the top of the pavement, and elsewhere. This is due to reduced lateral stresses and smaller suctions reducing the stiffness as defined by Equation 5.

Loss of strength, however, is not the direct cause, as Equation 3 relates to both natural and recompacted materials; it is however a factor in reducing the OCR and hence the lateral stress. Very similar results would have been expected using Equation 7.

Reducing the amount of unsaturated recompacted clay (compare Cases 2 and 3) has little effect but getting rid of all disturbed materials (Case 5) does reduce deflections considerably. The implication of this is that any of the disturbed material beneath the pavement could be undesirable. If the removal of disturbed, unsaturated clay materials cannot be assured then, for the same ground water conditions, there need be no difference between design for cuttings and embankments.

Use of empirically-based methods results in much reduced estimates of deflection on soft ground (compare Cases 6 and 7) principally because of inaccuracies in the M_r - CBR relationship at low CBR values (Brown et al., 1990). The stiffness values computed - even when giving comparable deflections (compare Cases 1 and 7) - are very high. This might be misleading.

All the analyses have been carried out assuming that equilibrium suctions apply. In fact most overconsolidated clays, when compacted, will generate excess negative pore pressures thus increasing the stiffness temporarily (Equations 3 and 5). Indeed the observation that subgrades often deteriorate considerably after construction may be due to suction equilibrium being slowly reached in recompacted and unsaturated soils. For design purposes the importance of characterizing a soil in its recompacted and unsaturated state is clear. Failure to do so may well result in inappropriate stiffness values being given to the layers of soil in a pavement. In the short-term there is potential for being more efficient in pavement design by taking temporary pore pressure reductions due to compaction/ trafficking into account although low plasticity clays such as the one used in these computations would be susceptible to rapid changes in behaviour due to environmental wetting and drying. For the long-term the equilibrium of unsaturated compacted soil will be the most important condition of the soil for design purposes.

The general constitutive equation for soils at a high, but not full, saturation level and typical temperate pavement suction conditions (Equation 7) provides a simple method by which long term stiffness may be estimated. The layered elastic method, used iteratively, can enable sensitivity to condition and soil property to be rapidly and conveniently studied.

7. CONCLUSIONS

a) Remoulding and de-saturation of clay - either when used as fill or because of trafficking - has important consequences for the deflection of a pavement under construction.

b) Analysis of data reported by Loach from triaxial testing carried out on compacted soil has enabled a new model for the stress-strain relationship of recompacted soil at high saturation levels and modest suctions to be developed.

c) It has beeen observed that many soils exhibit a finite maximum stiffness when tested at low deviator stresses. For pavemant design purposes a maximum value of 100 kPa is recommended. A more accurate value may be considered using models proposed for earthquake analysis but the model offered here should be appropriate for low confining stresses.

d) There is a clear need to collect suction data on clay soils immediately after reworking by compaction plant.

e) Simple layered elastic analysis can be used to compute behaviour of soils which are described by non-linear models.

f) Deflections of a pavement subjected to construction trafficking are sensitive to water table position and to disturbance of the clay on which it rests. They are not sensitive to the thickness of the disturbed soil.

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