MOISTURE CONDITIONS ASSOCIATED WITH PAVEMENTS IN SOUTHERN AFRICA

by

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DECLARATION

I, Frank Johannes Haupt, hereby declare that this dissertaion is my own work and that I have received no assistance other than that which I have acknowledged. The work was carried out while I was employed by the National Institute for Transport and Road Research and I have not submitted it to any other University.

F.J. HAUPT

ABSTRACT

A variety of geotechnical and pavement engineering problems require that one know what the eventual moisture conditions under a covered area will be, as considerable savings may result from preventive design measures. The historical development of the theory describing soil water movement is summarized and the factors affecting the establishment of the moisture regime in pavements are discussed. A comprehensive literature survey, firstly to determine the state of the knowledge concerning pavement moisture conditions and secondly to accumulate the greater part of available moisture prediction techniques, was carried out and the salient features are highlighted. It was decided to evaluate these models' prediction accuracies when applied to local conditions and for this all available local moisture information was gathered. The most common methods available to measure soil water potential are briefly discussed and recommendations regarding routine measurements put forward. None of the models tested were found to be applicable without modifications and consequently an exhaustive statistical analysis of the local data was undertaken in order to develop more accurate empirical prediction techniques for local conditions. This analysis included multiple stepwise linear regressions, the transformation and combination of certain predictors, linear regressions for certain ranges of values, non-linear regressions and linear regressions on only selected parameters. Conclusions about general moisture trends have also been drawn. The influence of climate and compactive effort on the empirical prediction models were evaluated and recommendations put forward as to how these influences may be accounted for in design. A method is proposed for calculating a probable maximum (or minimum) moisture content for design purposes. The term "characteristic maximum (or minimum) moisture content" is defined as that moisture content above (or below) which only a certain percentage of actual moisture content values will fall. The application of this concept in pavement and geotechnical engineering is explained. Relationships between other soil engineering parameters are also given. A provisional rational method, incorporating soil suction, is also proposed for completeness' sake. This method is intended, at least initially, to serve as a check on the empirical method, but with time it is bound to be refined to a more accurate

ABSTRACT (continued)

method. Preliminary work done to determine the seasonal and diurnal temperature and suction variations together with the associated pavement response variations, is reported on and it is concluded that more work should be done in this regard.

This dissertation is dedicated to two wonderful people,
Cornelis and Bettie Haupt

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CHAPTER 1

INTRODUCTION AND AIMS

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1. INTRODUCTION AND AIMS

1.1 Objectives

One of the main factors determininr [illegible]rformance of a flexible pavement is the shear strength of th[illegible]e. The majority of existing design procedures for flexible highway pavements are based on some measure of this property. Of the many parameters that affect shear strength, moisture content is very important and the question arises at what moisture content must this property be characterised for design purposes. Traditionally it has been measured in the "saturated" condition of the soaked CBR test.

Some contend tha[illegible]is approach is unrealistic or over-conservative in arid and semi-arid regions, as well-drained pavements in these areas may never become saturated. Although the U.S. Corps of Engineers carry out the CBR test at 4 days' soaking, they specify a rule of thumb reduction (1958) of 20 per cent in the thickness required un[illegible]r certain circumstances. McLeod (1953) has shown that the in-situ CBR meas[illegible] under existing pavements can be as high as 3 to 4 times the corre[illegible]ding 4 days soaked laboratory CBR. This has led the Canadian Depart[illegible] of Transport to reject this soaking for 4 days, basing their desi[illegible] [illegible]lely on field CBR without soaking. Some Israeli researchers (Livn[illegible] [illegible] Ishai, 1975) felt that, although the 4 days soaking procedure may [illegible] over-design, the Canadian Department of Transport procedure may l[illegible] under-design and they suggested a rational determination of an equilibrium moisture content. The Road Research Laboratory in London (1970) recommended that the subgrade material should be compacted in the laboratory at the equilibrium moisture content expected below the pavement without saturation. Some road authorities in Australia (Departments of Main Roads in New South Wales,

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1. INTRODUCTION AND AIMS

1.1 Objectives

One of the main factors determining the performance of a flexible pavement is the shear strength of the subgrade. The majority of existing design procedures for flexible highway pavements are based on some measure of this property. Of the many parameters that affect shear strength, moisture content is very important and the question arises at what moisture content must this property be characterised for design purposes. Traditionally it has been measured in the "saturated" condition of the soaked CBR test.

Some contend that this approach is unrealistic or over-conservative in arid and semi-arid regions, as well-drained pavements in these areas may never become saturated. Although the U.S. Corps of Engineers carry out the CBR test at 4 days' soaking, they specify a rule of thumb reduction (1958) of 20 per cent in the thickness required under certain circumstances. McLeod (1953) has shown that the in-situ CBR measured under existing pavements can be as high as 3 to 4 times the corresponding 4 days soaked laboratory CBR. This has led the Canadian Department of Transport to reject this soaking for 4 days, basing their designs solely on field CBR without soaking. Some Israeli researchers (Livneh and Ishai, 1975) felt that, although the 4 days soaking procedure may lead to over-design, the Canadian Department of Transport procedure may lead to under-design and they suggested a rational determination of an equilibrium moisture content. The Road Research Laboratory in London (1970) recommended that the subgrade material should be compacted in the laboratory at the equilibrium moisture content expected below the pavement without saturation. Some road authorities in Australia (Departments of Main Roads in New South Wales, 1974, and Western Australia, 1960) include the influence of the environment in their design techniques by either predicting the equilibrium moisture content or by reducing the required thickness under certain circumstances. During the Conference on Highway Engineering in Africa in Addis Ababa in 1974 Parker et al stated that there is an increasing tendency to determine the maximum moisture content that is likely to develop in the subgrade and to design the thickness of the pavement on the basis of the strength of the subgrade at this moisture content. Recent tests in South Africa, using the Heavy Vehicle Simulator (HVS), indicated that thin pavements behave much better than present theoretical models predict. One pavement has been found (Maree, 1979) to withstand between two and four times the design load (E80's) before any sign of distress occurred. It is believed that this is largely due to the fact that pavement layers, and the subgrade in particular, never get saturated under certain circumstances, while the design methods are based on the assumption of saturation. This worldwide recognition that the assumption of saturation is over-conservative in certain dry climatic regions, necessitates research into moisture conditions in South Africa as South Africa largely comprises these dry climates.

The second objection to the soaked CBR method is that it is an arbitrary test and that the design procedures based on it are largely empirical. It was developed in California in 1929 to be used in a design procedure which simply stated that, if the subgrade of a proposed pavement site were tested in the soaked condition, then the chances were statistically good that the pavement would perform adequately during its design life if a certain cover of suitable material were provided. The saturated state was implicitly related to the field moisture content through the data, as pavements in various stages of distress had been sampled and the design curves for cover requirements were based on this. This means that, if the original design curves are used, one is effectively designing for average moisture conditions in California in 1929, and Porter (1942) stated that "the soil often reaches the state of wetness and compaction comparable to the soaked specimen in the bearing test". This is clearly not applicable to southern African conditions.

Thirdly, with the progressive depletion of good quality road building material, the need arises to use lower quality materials and waste products. If these materials are to be used successfully, it is necessary to simulate their field behaviour theoretically. The development of mechanistic methods (Paterson, 1977; Walker et al, 1977) provides a tool for analysing pavement layers in various degrees of saturation and strength. To apply this successfully it is essential that a more rational method of moisture and strength characterization of the various layers be developed. This progress thus necessitates the development of techniques to predict the equilibrium moisture content of a layer (if this exists in South Africa), to predict the worst degree of saturation, and to predict the duration of the changes in moisture content in the various layers.

It was thus decided to carry out an investigation into moisture conditions in southern Africa, assuming, until proved otherwise, that equilibrium moisture conditions get established under sealed pavements as research elsewhere indicates (Redus, 1957; Russam, 1962; Aitchison and Richards, 1965). The main aims of this study may be summarized as follows:

(a) to investigate factors influencing pavement moisture conditions;

(b) to review available methods of predicting the equilibrium moisture content;

(c) to gather and utilize all pavement moisture data in southern Africa;

(d) to evaluate the various available empirical prediction techniques for southern Africa;

(e) to derive an improved empirical prediction technique for local conditions, if worthwhile;

(f) to attempt to validate rational prediction techniques, if possible, for local conditions;

(g) to make a preliminary investigation of the daily and seasonal variation in suction in pavement layers and to relate this to pavement performance.

(h) to propose, as a result of this study, a method of incorporating the influence of moisture rationally in pavement design.

1.2 Scope

As background to this dissertation the basic principles of moisture movement in a porous material and some factors affecting the establishment of a moisture regime are discussed.

The first step in the study was to review as many as possible of the existing moisture prediction models that have been developed in other countries. Data about the general moisture regime in the pavement were also gathered. The existing techniques can broadly speaking be classified into two categories:

(i) theoretical models, which seem to be the most accurate but whose application to southern African conditions would require considerable effort since relatively few local data are available;

(ii) empirical models, which are less accurate but require very little extra work to apply them to southern African conditions since they involve routine soil parameters, for which local data are readily available. Thus, these models could be readily tested for local conditions.

It was decided to give greater attention to the latter category.

In order to rank the existing models for local data, all the data available on moisture studies in southern Africa were accumulated. The models were then evaluated and ranked by testing them against the local data. This could only be done for the empirical models at this stage, as few data are available for the theoretical methods.

From this ranking it became clear that none of the existing models are readily suitable for local conditions and it was then attempted to improve or formulate models to suit southern African conditions. Empirical formulae were also developed to predict variables other than the equilibrium moisture content.

It was also found that the so-called rational approach could be applied to local conditions with some measure of success but it is felt that more work should be done in this regard.

A very important and significant phenomenon was discovered when it was observed that the daily variation in suction may be as large as the seasonal variation in certain regions and pavements. The significance of this to pavement rehabilitation and management is obvious.

To complete this study, a preliminary investigation into the influence of compactive effort and moisture on the CBR was carried out so that proposals could be put forward for pavement design.

CHAPTER 2

BACKGROUND INFORMATION

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2.1 Introduction

Seeing that this dissertation has, as one of its main aims, the prediction of pavement moisture conditions and because water and the movement of water is such a major factor influencing the performance of pavements and other surface structures, it is very important to understand the basic principles covering the movement of water through soil. The mathematical simulation of the flow of water through a complex porous material like soil is obviously very complex and it is not the aim here to indulge in this, but rather to concentrate on the physical principles involved. These principles, on which the mathematical models are based, are relatively simple and they will be explained in a simplified and summarized form here. The aim is therefore primarily to introduce the subject rather than dealing with it in detail. Secondly an attempt will be made to clear up any historical misconceptions about soil water and thirdly the basic mathematical models will be explained.

Once the principles governing the movement of water through a soil is clear, the factors affecting the establishment of a moisture regime will be discussed briefly.

Finally, the influence of water on the strength properties of a soil will be explained according to the classical effective stress theory, seeing that this aspect is the prime reason for the engineer's interest in water in soils.

2.2 Historical Development

In an effort to explain the movement and retention of water in the soil medium, Briggs developed the capillary tube hypothesis in 1897. He proposed that the prime reason for the retention of water in soil is the surface tension forces that develop as a result of the curved interfaces between the air and water. It logically follows that this retention is dependent on the number and size of the capillary cavities. According to this principle the soil water moves from the larger to the smaller cavities and the rate of movement is related to the curvature of the menisci, the surface tension and the viscosity of the water.

He proposed that soil water may be classified in three categories:

(a) Hygroscopic water, that is absorbed from an atmosphere of water vapour.

(b) Capillary water, that is kept in the capillary cavities through surface tension forces.

(c) Gravitational water, that drains out of the soil by gravity.

Several researchers refined this basic classification by adding further categories (Briggs and McLane, 1907; Widsoe and MacLaughlan, 1912). Bouyoucos (1921, 1936) proposed a new classification of soil water based on the results of his freezing point studies. His classification may be summarized as follows:

(a) Gravitational water: moves under the influence of gravity.

(b) Free water: freezed at 1,5 °C.

(c) Unfree water: does not function as a solute and is subdivided as:
capillary: freezes at -4 °C and is strongly absorbed;
compounded: does not freeze at -78 °C.

Keen (1920), Parker (1922) and others criticized this classification system widely, but it nevertheless emphasized the fact that a certain portion of the soil water is "strongly held" by the soil.

Zunker (1930) proposed the following comprehensive classification:

(a) Osmotic water: held in the cells of organic material (bacteria, etc.).

(b) Hygroscopic water: the quantity of water in the ground when no heat of condensation is released.

(c) Capillary water: kept in the fine pores of the soil and is linked with the water table.

(d) Held water: held to the surface of the soil particles by surface tension forces, but not linked to the water table. This category is subdivided into the following sub-categories:

(i) Film water: held around the particles like a skin.

(ii) Pore angle water: held in the angles formed by the contact points of the particles.

(iii) Capillary forced water: held in the capillary cavities, but is not linked to the water table.

(e) Gravitational water: moves downwards or horizontally in the zone of aeration. This category is also subdivided into the following sub-categories:

(i) Capillary gravitational water: moves downwards and laterally in the capillary pores.

(ii) Downward gravitational water: moves under the influence of gravity through the non-capillary pores to the water table.

(f) Groundwater: below the water table.

(g) Water vapour: in vapour form in the soil pores.

There are many more classification systems apart from the ones listed above. All of them are, however, variations of the basic capillary tube hypothesis originally proposed by Briggs.

Many objections may, however, be levelled against this hypothesis and this lead Buckingham, as early as 1907, to compare the flow of water through soil with the flow of electricity through a conductor. This marked the beginning of the application of the energy principle to water movement. It should be noted that, although there is no direct conflict between the two concepts in principle, more attention is now given to the energy concept. The main objection to the capillary tube theory is the arbitrary classification of the soil water. The fact remains that all soil water is subject to gravitation and is thus in a sense "gravitational" water.

In addition it is also true that the influence of capillarity does not start or end at certain degrees of wetness or at certain pore sizes. The only way to properly view the movement of water is to consider the total energy potential of the system. Initially the energy principle developed very slowly, but it gradually became more acceptable and today it forms the basis for the theory that explains soil water movement in porous media.

2.3 *The energy state of water in soil*

Considering the energy state of the water in soil, both kinetic and potential energy concepts are of importance. The kinetic energy can be expressed quantitatively as $\frac{1}{2}mv^2$, where m is the mass of the body and v is velocity. On an atomic and molecular scale all water above absolute zero is in motion and thus has kinetic energy. Bulk water also has kinetic energy by virtue of its motion but on this scale it plays a very minor part because the velocity is very small. On the macro scale, which will be considered here, potential energy plays by far the most important part and kinetic energy may be neglected. Although many soil water phenomena can be dealt with by considering only potential energy considerations, an important limitation is that it can only be applied to systems of a uniform temperature. As will be pointed out later, there is a well-defined temperature gradient in the top layers of the pavement and in this area kinetic as well as potential energy must be considered.

Potential energy is the energy that a body possesses by virtue of its position in a force field. Quantitatively it is expressed as the product of the force required to move the body against the force field and the distance it is moved.

Water in a porous medium like soil is subject to various force fields and with each force field a potential is associated. The earth's gravitational field exerts a vertical force upon the water. Force fields that are caused by the attraction of solid surfaces for water, pull water in various directions. Above the point under consideration, the weight of water, and sometimes of soil particles not supported by the soil matrix, also exert a vertical force on the water due to gravity (overburden pressure). Dissolved ions exert attractive forces on the water in all directions. The force associated with the attraction of water molecules for each other and the imbalance of these forces at air-water interfaces, constitute another important force.

This variety of forces and the different directions in which they act, make it extremely difficult to define force networks for the soil water. It is, however, possible to assess the potential associated with an increment of water. The potential energy differences between different points in the soil mass determine the direction of flow and this is the practical significance of describing soil water in terms of the energy associated with it.

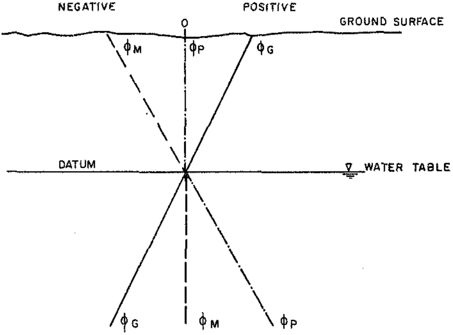
From the definition of the various potentials it is clear that for water at rest in a system with reference to the earth's surface, the sum of the potentials, or the total potential at a point, must be either constant or zero; this depends of course on the reference taken. The total potential may be defined as:

$$\phi_t = \phi_m + \phi_g + \phi_p + \phi_\pi + \phi_\Omega \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2.1)$$

where ϕ_t = total potential
ϕ_m = matric potential
ϕ_g = gravitational potentail
ϕ_p = pressure potentail
ϕ_π = osmotic potential
ϕ_Ω = overburden potential

2.3.1 Matric potential (suction)

Matric potential is the amount of work that must be done per unit quantity of pure water in order to transport reversibly and isothermally an infinitesimal quantity of water, from a pool containing a solution identical in composition to the soil water, at the elevation and the external gas pressure of the point under consideration to the soil water (Commission I, ISSS, 1963). It may be seen that matric potentials are negative in as much as water from the reference pool will flow freely to dry material with the subsequent release of energy in the form of heat. This means that work will be done during the process of wetting rather than that work is required as stated by the definition. The matric potential is therefore always negative above the water table and zero below it (Figure 2.1). This means that water will tend to flow spontaneously to the drier soil above the water table because the matric potential is lower there. This is because the matric potential is associated with the



NOTE: THE OSMOTIC POTENTIAL CAN TAKE ANY VALUE DEPENDING ON THE CONCENTRATION OF THE SOIL WATER SOLUTION

FIGURE 2.1

SOIL WATER POTENTIAL IN AN EQUILIBRIUM STATE

attractive forces between solids and water and between different water molecules. This phenomenon has historically been referred to as the capillary potential, because it is analogous over a significant part of its range to the situation that exists where water rises in small capillary tubes. Capillarity, however, is only associated with an air-water interface and the resulting forces, whereas the term matric potential is more general to include water-solid interfaces, for example the water held directly on the solid particles. his term is commonly called the suction of the soil. The measurement of suction can be performed in several ways but it is not the intention to elaborate on these methods here. Good summaries of available methods are given by Marshall (1959), Croney et al

(1952) and Richards (1968).

2.3.2 Gravity potential

Gravitational potential is the amount of work that must be done per unit quantity of pure water in order to transport reversibly and isothermally an infinitesimal quantity of water from a pool containing a solution identical in composition to the soil water at a specified elevation of the point under consideration (Commission I, ISSS, 1963). The elevation that is chosen as a datum determines whether the point under consideration has a positive or a negative gravitational potential. This gravitational potential is identical to any other gravitational phenomenon. If the datum is taken to be the water table, then the potential will be negative above it and positive below it. For an equilibrium situation the matric and gravitational potentials must be equal in magnitude but different in sign. This means that it is very easy to determine the matric potential in an equilibrium situation because it is equal to the gravitational potential which is simply measured by the height above a reference point. For such an equilibrium situation, with the datum at the water table, the matric and gravitational potentials will be equal and opposite above the water table but below the water table and gravitational and pressure potentials will be equal (Figure 2.1).

2.3.3 Pressure potential

The pressure potential is the potential due to the weight of the water above the point under consideration plus that due to the gas pressure if it is different to that at the reference level. This potential is zero at the water table and above it and is equal but of opposite sign to the gravity potential below the water table (Figure 2.1). It could also be referred to as the piezometric potential and it is equivalent to the piezometric head. In soil above the water table this is sometimes referred to as the pneumatic potential if the gas pressure is different to that at the water table. To prevent confusion, it should be stated here that most methods of measuring the matric potential actually use the pneumatic pressure to deduce the matric potential. In this case, the pressure potential is identical to the matric potential. For clarification, both potentials could be considered as the same potential which is called matric potential above the water table and pressure potential below the water table. The pressure potential as a result of vapour pressures may be measured with ordinary manometers, whereas the pressure potential as a result of water pressures may be measured with either manometers or piezometers.

2.3.4 Osmotic potential

Osmotic potential is the amount of work that must be done per unit quantity of pure water in order to transport reversibly and isothermally an infinitesimal quantity of water from a pool of pure water at a specified elevation at atmospheric pressure to a pool containing a solution identical in composition to the soil water (at the point under consideration) but in all other respects identical to the reference pool (Commission I, ISSS, 1963). This potential results from the hydration of the different ions in the soil solution. The polar nature of the water molecule causes positive and negative ions to orientate themselves around the water molecule, or vice versa, and the osmotic potential refers to the work required to pull water away from these ions.

The most common example of the presence of osmotic forces is the rise of water in the one leg of a U-tube that contains a solution in that leg and pure water in the other, separated by a semi-permeable membrane in the bottom. Pure water passes through the membrane because of the attraction of ions for the water molecules. The height difference between the water levels in the two legs is called the osmotic pressure.

These osmotic potentials are very important in membrane-like situations, for example the uptake of water by plant roots. Of some engineering significance is the fact that the air-solution interface acts like a membrane and the vapour pressure in the surrounding air is reduced proportionately in a comparable manner to the pressure difference developed across a solution membrane. Therefore osmotic pressures are involved wherever an air-solution interface exists. It also means that more energy must be supplied to remove water from solutions than from pure water. This means that if the "suction" is implicitly measured by using the vapour pressure, then the osmotic potential and the matric potential are included.

While discussing osmotic potentials, it is also interesting to note that the attraction clay particles have for water, gives rise to a potential that is sometimes classified as an osmotic potential. This is debatable, but fundamentally it should be considered a matric potential.

Osmotic potentials are similar to potentials described before wherever a membrane-like situation exists, and can be added or subtracted from such potentials to form a total potential. However, where a membrane situation does not exist, the ions diffuse spontaneously into the soil solution and in such circumstances the osmotic potential cannot be matched with any other potential.

2.3.5 Overburden potential

The overburden potential results from the pressure exerted by the soil above the point under consideration on the point under consideration. It is similar to the pressure potential exerted by the water above the point under consideration and is sometimes included implicitly in the pressure potential. Where this is done, the pressure and gravity potentials are no longer equal and of opposite sign and this fact often makes it desirable to recognise the overburden potential as a separate potential.

2.3.6 Combination of potentials

In civil engineering the potentials governing the flow in the saturated and unsaturated condition are of major importance. From Figure 2.1 it can be seen that certain potentials are important under certain flow conditions and all potentials do not have to be considered under all circumstances.

For unsaturated flow conditions the matric and gravity potentials are of greatest concern. There exists no commonly accepted term for the sum of these two potentails and most equations include both terms explicitly.

For saturated flow conditions the pressure and gravity potentials are of major importance. The sum of these potentials is referred to as the hydraulic potential (ϕ). The pressure potential usually includes any overburden, otherwise a separate term is included.

For flow conditions in the field the term head rather than potential is often used:

$$H_t = H_m + H_p + Z \qquad (2.2)$$

where H_t = total head
H_m = matric head
H_p = pressure head
Z = distance above datum

2.4 The flow of water in soil

2.4.1 Saturated flow

If soil could have been simulated by a series or bundle of thin tubes, the overall flow rate would have been the sum of the separate flow rates through the individual tubes. This would be easy, because existing theory can accurately predict the laminar flow rate through a cylindrical tube.

Unfortunately, soil does not resemble uniform, smooth tubes, but is highly irregular, tortuous, and interconnected. For this reason, flow through complex porous media is generally described in terms of a microscopic flow velocity vector, which is the overall average of the microscopic velocities over the total volume of the soil. The detailed flow pattern is ignored and the material is treated as a uniform medium.

Darcy (1856) developed an equation for this situation which relates the flux (discharge per unit cross-sectional area) to the hydraulic gradient and the hydraulic conductivity, viz:

$$q = K\frac{\Delta H}{L} \qquad (2.3)$$

where $q = \frac{Q}{A}$ = flux

K = constant (hydraulic conductivity)

ΔH = hydraulic head drop

L = length of soil column over which the head drop happens

Slichter (1898) generalized Darcy's equation into a three-dimensional macroscopic differential equation to include unsteady flow (i.e. flux changes with time:

$$q = -K\left(\frac{\partial}{\partial x}\frac{\partial}{\partial y}\frac{\partial}{\partial z}\right)H$$
$$= -K\nabla H \quad \ldots\ldots \quad (2.4)$$

Mathematically, Darcy's law is similar to the linear transport equations of classical physics, including Ohm's law (current is proportional to the electrical potential gradient), Fouriers law (heat conduction is proportional to temperature gradient) and Ficks law (diffusion porportional to concentration).

This hydraulic gradient can be resolved into pressure and gravitational potentials as defined earlier.

Darcy's law only applies as long as the flow is laminar. The linearity of the flux versus hydraulic gradient relationship fails at high flow velocities because then the inertial forces are no longer negligible compared with viscous forces (Hubbert, 1956). In fine-grained soils like silts and clays, laminar flow nearly always occurs but in coarse sands and gravels turbulent flow may take place. The quantitative criterion for the onset of turbulent flow is the Reynolds number, R_e

$$R_e = \frac{d\bar{V}\rho}{\nu} \quad \ldots\ldots \quad (2.5)$$

where $\bar{V}$ = mean flow velocity

d = effective pore diameter

ρ = liquid density

ν = liquid viscosity

Turbulence sets in when this number is between 1000 - 2500 (Scheidogger, 1957; Childs, 1969) in straight tubes. In soils where the "tubes" are curved it is only safe to assume laminar flow as long as the Reynolds number is less than unity (Hillel, 1973). Deviations from Darcy's law may also occur at the opposite end of the flow velocity range, namely at low gradients and in small pores. Some investigators (Swartzendruber, 1962; Miller and Low, 1963; Nerpin et al, 1966) claim that, in this region, low hydraulic gradients cause only low flow rates that are less than proportional to the gradient. This is explained in terms of the so-called "poly-water" effect (Lippincot et al, 1969) and, because it has little practical significance, will not be discussed here.

In order to develop a general flow equation, the Darcy equation must be combined with the general equation of continuity which states that the inflow minus the outflow must be equal to the change of storage of a soil volume. Considering a simple one-dimensional case, with q_x being the flux in the x-direction, the rate of increase of q_x with x must equal the rate of decrease of volumetric water content θ with time t:

$$\frac{\partial \theta}{\partial t} = -\frac{\partial q_x}{\partial x} \qquad (2.6)$$

which, in multidimensional systems, becomes:

$$\frac{\partial \theta}{\partial t} = -\left(\frac{\partial q}{\partial x} + \frac{\partial q}{\partial y} + \frac{\partial q}{\partial z}\right)$$
$$= -\nabla \cdot q \qquad (2.7)$$

Recalling Darcy's equation:

$$q = -K\left(\frac{\partial H}{\partial x}\,\frac{\partial H}{\partial y}\,\frac{\partial H}{\partial z}\right)$$
$$= -K\nabla H \qquad (2.4)$$

and combining this with equation 2.7, we obtain the general flow equation:

$$\frac{\partial \theta}{\partial t} = K\left(\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} + \frac{\partial^2 H}{\partial z^2}\right)$$
$$= \nabla \cdot K\nabla H \qquad (2.8)$$

This is the general flow equation and by considering steady flow conditions ($\frac{\partial \theta}{\partial t} = 0$), it can be reduced to the Laplace equation:

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} + \frac{\partial^2 H}{\partial z^2} = 0 \qquad (2.9)$$

Thus, for the proper description of water flow, two of the following three parameters must be specified and the third may then be calculated: flux, hydraulic gradient and conductivity. The hydrailic conductivity (permeability) at saturation is a characteristic property of a soil, and is related to porosity and pore size distribution. All attempts, to date to relate permeability to porosity, pore size distribution, interval surface area, etc. have met with disappointing results and it is usually best to measure it directly or use a statistical process to estimate it (Scheidegger, 1957; Carman, 1939).

2.4.2 Unsaturated flow

Many situations of engineering interest occur when the soil is in an unsaturated condition. The unsaturated flow processes are generally much more complicated than the saturated flow conditions because they often entail changes in the state and content of the water. Such changes involve complex relations among water content, suction and conductivity, which may be affected by hysteresis. In recent years the development of high-speed computers has paved the way for practical application of numerical methods to these problems and significant advances have been made. Only the basic principles underlying the models that simulate unsaturated flow will be highlighted here.

The moving force in saturated soil is the hydraulic gradient, which is a positive pressure gradient, but in the case of unsaturated soil the moving force is a negative pressure or suction gradient, as explained in Figure 2.2. Another main difference between saturated and unsaturated flow lies in the hydraulic conductivity. When a soil is saturated, all the pores are filled with water and the conductivity is maximal. As the soil dries out, some pores become airfilled and this drastically reduces the conductivity, because the conductive cross-sectional area is reduced. It is also true that the largest pores empty first, leaving only the smaller pores to conduct the water along longer and more tortuous channels. (A 1 m radius pore will conduct the same as 10 000 pores of radius 0,1 mm.) It is a characteristic of coarse-grained materials that they possess a high conductivity when saturated because of the large pores, but when desaturation sets in, the large pores empty first and the conductivity may drop to zero if the water films become discontinuous. Because the conductivity sometimes drops so low, very high suctions (thousands of times greater that gravity) are necessary to cause any appreciable flow. Th... ans that although, when saturated, sand conducts water more rapidly than a clay, the direct opposite may be true when the soils are unsaturated. Thus, in practical terms, a sand layer may act as a barrier to water under certain circumstances of desaturation.

The problem in modelling unsaturated flow lies in the fact that although a steady flow situation may be set up, the suction gradient, the conductivity and the moisture content will not vary linearly along the flow line. The conductivity must be calculated for each point along the flow line. In general, if the suction is high, the conductivity and moisture content will be low. The conductivity does not only vary with suction, but it also varies with material type. No fundamentally based equation of general validity is available for the relation of conductivity to suction or moisture content, and existing knowledge does not allow the reliable prediction of unsaturated conductivity from basic soil properties. Various empirical relations have been proposed (Gardner, 1960) and they are used in the various models after experimentally determining which relation gives the best fit for that soil. The relation between suction and conductivity is also dependent on hysteresis.

Richards (1931) extended the Darcy equation to include unsaturated flow conditions, with the provision that the conductivity is now a function of the matric suction head, i.e. $K = K(\phi)$

$$\therefore\ q = -K(\phi)\left(\frac{\partial H}{\partial x}\ \frac{\partial H}{\partial y}\ \frac{\partial H}{\partial z}\right) \qquad (2.10)$$
$$= -K(\phi)\nabla H$$

where H is the hydraulic head gradient which may include both suction and gravitational components. This equation does not make provision for the very strong influence hysteresis has on soil-water characteristics. It can only be applied for either the wetting or the drying cycle. Because the relation of conductivity to moisture content $(K(\theta))$ is much less effected by hysteresis, equation 2.10 may be rewritten as

$$q = -K(\theta)\left(\frac{\partial H}{\partial x}\ \frac{\partial H}{\partial y}\ \frac{\partial H}{\partial z}\right)$$
$$= -K(\theta)\nabla H \qquad (2.11)$$

which still leaves the problem of hysteresis between ϕ and θ unresolved.

Introducing the continuity equation

$$\frac{\partial \theta}{\partial t} = -\left(\frac{\partial q}{\partial x} + \frac{\partial q}{\partial y} + \frac{\partial q}{\partial z}\right)$$
$$= -\nabla \cdot q \qquad (2.12)$$

we get

$$\frac{\partial \theta}{\partial t} = K(\phi)\left(\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} + \frac{\partial^2 H}{\partial z^2}\right)$$
$$= \nabla \cdot (K(\phi)\nabla H) \qquad (2.13)$$

Because the hydraulic head is usually the sum of the suction (ϕ) and gravitational (z) heads we can write

$$\nabla H = -\nabla(\phi - z)$$

thus equation 2.13 becomes

$$\begin{aligned}\frac{\partial\theta}{\partial t} &= -\nabla\cdot\{K(\phi)\ \nabla\ (\phi - z)\} \\ &= -\nabla\cdot\{(K(\phi)\frac{\partial\phi}{\partial x}\ K(\phi)\frac{\partial\phi}{\partial y}\ K(\phi)\frac{\partial\phi}{\partial z}) - (K(\phi)\frac{\partial z}{\partial x}\ K(\phi)\frac{\partial z}{\partial y}\ K(\phi)\frac{\partial z}{\partial z})\} \\ &= -\nabla\cdot(K(\phi)\nabla\phi) + \frac{\partial K(\phi)}{\partial z} \\ &= -\frac{\partial}{\partial x}(K(\phi)\frac{\partial\phi}{\partial x}) - \frac{\partial}{\partial y}(K(\phi)\frac{\partial\phi}{\partial y}) - \frac{\partial}{\partial z}(K(\phi)\frac{\partial\phi}{\partial z}) + \frac{\partial K(\phi)}{\partial z} \quad \ldots\ldots\ (2.14)\end{aligned}$$

This is the general flow equation, but in many flow conditions Δz is negligibly small in comparison to the high suction gradients, so

$$\frac{\partial\theta}{\partial t} = \nabla\cdot\{K(\phi)\ \nabla\ (\phi)\} \quad \ldots\ldots\ (2.15)$$

or for one-dimensional flow

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial x}\{K(\phi)\frac{\partial\theta}{\partial x}\} \quad \ldots\ldots\ (2.16)$$

Although the concepts explained here are relatively simple, the equations have to be used with considerable care because of the influence of hysteresis, vapour movement and the unpredictable relations between suction and conductivity.

To simplify matehmatical and experimental treatment, several techniques have been adopted, but they fall outside the scope of this dissertation (see Childs and Collis-George, 1950, for the diffusivity concept and Klute, 1952; Philip, 1955, 1957, for Boltzmann transformation). It should however be noted that the main obstacle preventing the development of accurate theoretical moisture prediction models, lies in the difficulty in predicting the input values accurately.

2.4.3 Vapour and solute movement

In general the majority of water that moves through soil is in the liquid form and the entire body of fluid flows in response to pressure differences. Water vapour can, however, move through the soil by diffusion. Water vapour is always present in an unsaturated soil and vapour diffusion occurs whenever a vapour pressure difference develops. The diffusion equation for water vapour is

$$q_d = -D\frac{\Delta P}{L} \qquad (2.17)$$

where q_d = diffusion flux

D = diffusion coefficient for water vapour (lower than for open air)

ΔP = vapour-pressure difference between 2 points a distance L apart.

In a non-saline soil at constant temperature vapour-pressure differences are negligibly small. Dissolved salts and temperature differences can cause large vapour-pressure gradients which induce diffusion. As the temperature increases, so the vapour-pressure increases and water moves from a hot to a cold part of the soil. Water thus moves downward during the day and upward during the night. Temperature gradients can also induce liquid flow (Hillel, 1973).

It appears impossible to separate absolutely the liquid from the vapour movement as the rate of vapour movement often exceeds the predicted diffusion rate (Philip and De Vries, 1957; Cary and Taylor, 1962). Soil water is invariably a solution and the solutes can only exist in the liquid form.

Comprehensive reviews of the interactions of solutes in soils were published by Gardner (1965) and Bresler (1970) and it will suffice here to say that dissolved substances can move by molecular diffusion, due to concentration of gradients within the solution, or by convection, due to the mass flow of the soil water.

2.5 Factors affecting the establishment of the moisture regime in pavements

Remembering the principles explained earlier in this chapter, it is clear that the only way in which changes in the moisture content of subgrades can occur is by movement of water from areas of higher total potential to areas of lower potential. This may occur under different physical conditions as indicated in Figure 2.2.

(a) By the seepage of water into the subgrade from higher ground adjacent to the road.

(b) By a rise or fall in the level of the water table.

(c) By the percolation of water through the surface of the road.

(d) By the transfer of moisture either to or from the soil in the shoulders as a result of differences in moisture content.

(e) By the transfer of moisture to or from lower soil layers.

(f) By the transfer of water vapour through the soil.

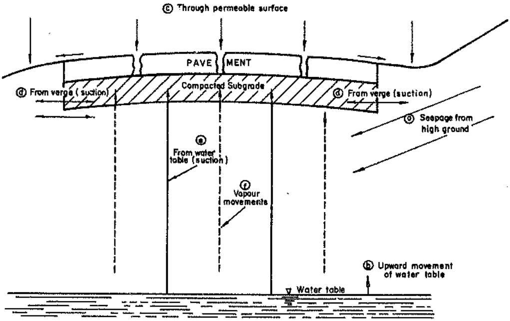


FIGURE 2.2

WAYS IN WHICH WATER CAN ENTER AND LEAVE ROAD SUBGRADES

All these ways of water movement are simply examples of the general principle that water moves from a high to a low potential.

The moisture regime in an exposed road pavement depends mainly on the following factors (NAASRA, 1974).

(a) General drainage condition:

Position of catch drains, table drains and subsoil drains
Shoulder cross-fall
Longitudinal grade
Vegetated shoulders, sealed shoulders
Formation profile
Cut or fill

(i) Position of the water table:

Static
Variable
Deep
Shallow

(c) Climate:

Rainfall
Evaporation
Temperature
Thermal gradients

(d) Construction:

Boxed or trenched
Untrenched
Moisture at compaction and sealing

(e) Surrounding landform:

Drainage depressions, swamps
Adjacent rivers, irrigation areas, etc.
Vegetation
Run-off and permeability of soil strata

(f) Subgrade, shoulder and pavement materials:

Grain size
Pore size
Density
Mineralogy
Shrink-swell properties
Permeability
Salinity

(g) Vegetation:

(Closely related to climate)

(h) Road condition:

Permeability of the surfacing

When considering factors influencing the moisture regime in a specific area, it is important to recognise 2 distinct situations. Firstly, if the water table is close to the surface, it is the main factor influencing the moisture regime. Shallow is defined as not more than 3 m in sands, 6 m in silts or 10 m in clay (Russam, 1962). If the water table reaches these depths, all other factors become insignificant. Secondly, if the water table is deeper than these depths, other factors, such as climate, become predominant and they determine the moisture conditions. This category is again subdivided in two categories, viz non-arid and arid climates. In a non-arid (Richards, 1967; O'Reilley et al, 1968) climate the moisture content is determined by the conditions below the zone of seasonal variation and in an arid (Richards, 1968) climate it is determined by atmospheric humidity.

2.6 The effective stress principle

The basic principle of the effective stress law will be explained here, but its incorporation into shear strength and volume change laws will not be discussed. Bishop (1960) defined the effective stress principle for saturated soils as follows:

(a) the change in volume of an element of soil depends not on the change in total normal stress but on the difference between the change in total normal stress and the change in pore pressure.

(b) the maximum resistance to shear on any plane in the soil is a function not of the total normal stress acting on the plane, but of the difference between the total normal stress and the pore pressure.

Thus

$$\sigma' = \sigma - u \quad \ldots\ldots \quad (2.18)$$

where σ' = effective normal stress
σ = total normal stress
u = pore pressure

This equation holds for conditions where the pore space is filled with a single fluid. The pore pressure in saturated soils is generally positive, but negative pressures have been measured and the validity of equation 2.18 demonstrated in both conditions.

Where there are two fluids in the pore space at different pressures due to surface tension, as in a partly saturated soil, equation 2.18 must be generalized to include both pressures. Bishop (1955) proposed the following equation

$$\sigma' = \sigma - u_a + \chi(u_a - u_w) \quad \ldots\ldots \quad (2.19)$$

where u_a = air pressure in the pore space
u_w = water pressure in the pore space
χ = factor depending on degree of saturation, soil type, etc.

It is common to assume that the air in the pore space is at atmospheric pressure while the water pressure is negative. In this case equation 2.19 becomes

$$\sigma' = \sigma + \chi u \quad \ldots\ldots \quad (2.20)$$

(put $u_w = u$)

This is the simplified effective stress law for unsaturated conditions where no external stress is present. The χ-factor introduced by Bishop is equivalent to Aitchison's ϕ-factor (1960), Jennings' β-factor (1960), and Croney et al's β'-factor (1958). This negative pore water pressure may be equated to the soil suction in cases where no external pressure is carried by the soil water.

The negative pore water pressure u is related to the soil matric suction by the equation 2.21

$$u = \phi_m + \alpha\sigma \qquad (2.21)$$

where σ = total applied stress

ϕ_m = soil matric suction

u = negative pore water pressure

α = change of negative pore water pressure with applied pressure at constant moisture content

This shows that if a pressure is applied to the soil and the moisture content does not change, the suction remains constant while the pore water pressure changes.

This short summary relates suction to negative pore water pressure, and indicates how it may be incorporated in the effective stress law. These principles may be used in predicting the volume change and shear strength performance of soils.

More work is needed to determin the influence of other water potentials, especially osmotic potential, on the shear strength of the soil. As pointed out earlier in this chapter, the osmotic potential may be added to the other potentials in membrane-like situations, but not where no membrane exists. Its influence on shear strength will also not be the same in these two categories.

2.7 Conclusions

The following conclusions may be drawn from this chapter.

(a) The historical view of classifying soil water into categories is misleading and complicates the understanding of soil water movement.

(b) It is easier and more correct to consider the energy potentials causing moisture movement.

(c) For unsaturated flow, only insignificant errors will be made if only the matric and gravity potentials are considered.

(d) Although the mathematical theory describing the movement of water through a porous medium is well developed, and may be readily modelled in a finite element program, the input values remain a problem. For example, how can the relationship between suction and conductivity for any material be accurately modelled?

(e) There is no way that water can move from one point to another if a potential gradient does not exist and, conversely, water will continuously move if a potential gradient exists until the gradient is dissipated. This means, for example, that a sand and a clay layer in contact with each other may be in moisture equilibrium with widely different moisture contents.

(f) If the water table is "close" to the surface, it is the major factor influencing the moisture regime; if it is "deep", climate becomes the major factor.

(g) The matric potential in a soil is related to the negative pore water pressure through the addition of the fraction of applied pressure taken up by the pore water. In this way it may be incorporated directly into the effective stress law.

(h) More work is needed to determine the influence of total water potential (especially osmotic) on shear strength.

CHAPTER 3

SURVEY OF EXISTING KNOWLEDGE

CHAPTER 3

SURVEY OF EXISTING KNOWLEDGE

CONTENTS

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CHAPTER 3

SURVEY OF EXISTING KNOWLEDGE

3.1 Introduction

It is the aim of this chapter to briefly summarize the salient features regarding the moisture regime in pavement subgrades, that emerged from a comprehensive literature survey. The aim of this literature survey was to determine the state of knowledge concerning the factors influencing the establishment of a moisture regime, and secondly, the models available to predict equilibrium moisture conditions where these exist.

3.2 Review of factors influencing a moisture regime

Several agencies and individuals have conducted extensive subgrade moisture studies in past years. Comprehensive summaries (Marks and Haliburton, 1967) of these studies are available and only the main features will be highlighted in this dissertation.

Most studies of moisture conditions in pavement systems indicate that the moisture content varies with season, climatic conditions, geographic location and the type of pavement system. Concluding from a study of 13 USA airfields located in non-frost susceptible areas, Redus (1957) indicated that moisture conditions may go up or down, following construction, but appear to stabilize after about 2 years, subsequently displaying only small fluctuations. Low and Lovel (1959) concluded from their study of current literature that the moisture content shows continuous, if small, variations with the seasons. In a long-term study of moisture contents beneath rigid pavements in Missouri, Guinee and Thomas (1955) found that moisture variations in the top levels of the sub-base and subgrade were greater than those noted at deeper levels. Chu and Humphries (1971) found that moisture conditions in pavement systems in Carolina varied with season, soil type and location in the pavement system. They found that the subgrade moisture content was influenced by the depth of the groundwater table.

Aitchison and Richards (1965), drawing from results on their study of moisture conditions in pavement subgrades throughout Australia, stated that moisture stability beneath the greater part of a paved area occurred at every test site, even though the climatic conditions at the sites were widely different. This stable moisture regime was largely beyond engineering control because of the dominant influence of the environment. Moisture conditions at the verges or under the edges of the seal were found to be variable and largely dependent on engineering design and construction. They found that, after heavy rain, the profile beyond the seal wets rapidly to considerable depth and remains wet for periods of a few weeks. Only a slight penetration of water under the seal was noted, but the greater part of the subgrade was unaffected. (They only looked at average and better seals.) Good surface drainage reduced the deep seasonal moisture variation adjacent to the seal. This investigation also showed that there was a general agreement between Thornthwaite's Moisture Index and the suction in the stable zone beneath the pavement. They proposed that this may be used as a general method to determine design suction values.

A field study conducted by the Corps of Engineers (1958) indicated that the amount of precipitation may have considerable influence on the moisture conditions in airport subgrade and base materials. De Bruijn (1965) found that for various locations in South Africa the moisture content under the pavement was significantly higher than the moisture content in the open field. He could not determine the depth of influence of the pavement.

Marks and Haliburton (1969), drawing from a short-term study of typical Oklahoma highways, concluded that moisture variations occur beneath highway pavements in an annual cycle with maximum moisture contents occurring during the winter months (summer rainfall region). Hicks (1948) found that for 20 stations in North Carolina the moisture contents of the bases and sub-bases were highest in late winter or early spring (summer rainfall regions). Woodman et al (1963) found that in Rhodesia the wettest subgrade values occurred during and subsequent (January/March) to the "main rains with a gradual drying out during the dry season".

Not all researchers agree on the extent to which precipitation influences the variation in subgrade moisture content. Kubler (1963), after analysing many data on subgrade moisture content and precipitation for West Germany, could not establish a relationship between precipitation and change in subgrade moisture content. Marks and Haliburton (1969) indicated that cyclic moisture content variations were considerably affected by precipitation at sites where the pavement was poor (greater degree of cracking and perviousness in the pavement surface). Moisture content changes could not be correlated with precipitation for pavement sections with high ratings (little cracking, good surface conditions). The moisture content changes for the higher rated pavements were primarily attributed to temperature effects, although these effects were generally small (Osterhout and Haliburton, 1969).

Haliburton (1971) found subgrade moisture accumulation and subgrade moisture variation to be the two basic types of moisture behaviour in Oklahoma subgrades (compare equilibrium subgrade moisture conditions). He found that moisture contents of subgrades under impervious pavements increased to approximately 1,1 to 1,3 times the plastic limit within the first two years of construction. Moisture variations were, however, observed after the equilibrium point had been reached and he therefore regards the work equilibrium as a misnomer. At the same time in Australia, Richards et al (1970) found that the subgrade equilibrium moisture content will be considerably drier than optimum moisture content for Std. AASHTO compaction.

The results of an analysis of the influence of precipitation on soil moisture content by Moulton and Dubbe (1968) showed that the amounts of precipitation occurring at various periods prior to moisture content sampling were not statistically significant, at the 5 per cent level, in explaining the observed variations of moisture contents, either in the base and sub-base materials or in the subgrade soils. They found that the moisture content in granular base and sub-base materials was more dependent upon the drainage characteristics of the material and the site than upon precipitation.

Observations by Turner and Jumikis (1956) showed that moisture conditions beneath a pavement are affected by the groundwater table. They found that precipitation could modify the position of the water table and subgrade moisture content and that the degree of change was sensitive to the type of precipitation. They found that more water from melting snow precipitation percolated into the ground than if the precipitation was in the form of rain.

Coleman (1965) reported that the seasonal moisture regime in the verge and foundation of a road is determined by the soil type (geology), climate and vegetation, the order being that of decreasing importance. This has not been borne out for conditions other than British.

Several investigators (Redus, 1957; Low and Lovell, 1959) have concluded that moisture contents at the pavement edges are generally higher than those at the interior locations. Guinee and Thomas (1955) pointed out that water enters the pavement more easily and in greater volumes at the pavement edges. Benkelman (1959), in an analysis of WASHO Road Test deflection data, correlated inner and outer wheel track deflections with degree of subgrade saturation and found that adverse moisture contents exist at pavement edges. Aitchison and Richards (1965) also noted the greater moisture content fluctuations at the pavement edges for Australian pavements.

Woodman et al (1963) also found that seasonal wetting, similar in extent to that in the unpaved area, occurs in the shoulder and that this influences the subgrade near the edge of the road. The wettest values occur during and subsequent to the main rains. Wallace and Leonardie (1978) reported that in the coastal north of Queensland in Australia there is a strong seasonal fluctuation in pavement performance which they could directly relate to seasonal moisture changes. The distress was strongly related to the wet season and was caused by water entering the pavement and subgrade. It should be noted that the water table was close to the surface in this case. Vaswani (1975) reported that in Virginia, following construction, subgrade moisture increased rapidly due to precipitation. The final level of moisture content depended on density, compaction and soil grading. He also reported a noticeable moisture variation due to temperature variations. After about 10 years, there was practically no change in subgrade moisture and temperature gradient had no effect.

Moisture content, dry density, CBR and plate bearing data were obtained for flexible pavements at the AASHTO Road Test. These data indicate that CBR decreases with increasing moisture content. It was found that the dry density did not vary significantly with moisture content changes.

Kersten (1945) noted a slight increase in moisture content with an increase in depth. Data from Marks and Haliburton (1969) suggest a rather erratic moisture content-depth relationship. Depending on the nature of the soil profile beneath the pavement, it is possible to have wide variations in field moisture content because of the discrepancies in equilibrium moisture-tension relationships for different soils.

Kersten (1944) summarized extensive field data for the top 150 mm of the subgrade under flexible pavement systems as follows:

(a) The degree of saturation existing in the subgrades of numerous projects averaged 73 per cent; the range being from 60 - 80 per cent. Fifteen per cent of the tests showed a saturation value of 90 per cent or higher.

(b) The subgrade soils of projects in which a high average percentage of saturation occurred were in most instances either clay or silty clay.

(c) Subgrade moisture expressed as a percentage of plastic limit averaged 77 per cent, the range being from 64 - 82 per cent. Seventeen per cent of the tests showed moisture contents in excess of the plastic limit.

(d) Fine-textured soils, such as clays, exhibited a marked tendency to attain moisture contents in excess of their plastic limit. Sandy loams rarely had moisture contents as great as their plastic limit. Loessial silty soils tend to attain moisture close to their plastic limit.

(e) The optimum moisture content of the soils were exceeded by the field moistures in about 1/3 of the determinations reported.

(f) A general trend when comparing moisture content with saturation or plastic limits is that clays have high degrees of saturation and the moisture content generally exceeds the plastic limit. Sand on the other hand has low degrees of saturation and is generally drier than the plastic limit.

(g) Only slight changes of moisture content occurred for periods from one to five years.

(h) Sandy materials are affected more by annual precipitation than are clayey materials.

(i) The moisture content of about 25 per cent of the sands, 70 per cent of the sandy loams, and 90 - 100 per cent of the heavier soils exceeded the optimum moisture content.

(j) Moisture conditions for distinctive soil areas were generally quite uniform and well-defined.

(k) Comparisons of the moisture contents in similar soils beneath rigid and flexible pavements on the same airfield showed, in most instances, higher values beneath the concrete; on the average the percentage of saturation differed by about 10 per cent for the two types of pavement.

(l) Studies of moisture variations with depth in the subgrade for arid regions were not conclusive but indicated only slight average variations in the upper 1 m.

Concluding from an extensive study of moisture contents under flexible airfield pavements, Redus (1957) made the following observations:

(a) In-situ moisture contents varied directly, although erratically, with the percentage finer than 0,075 mm.

(b) Some subgrade moisture contents exceeded the plastic limit but all were below the liquid limit. All sub-base moisture contents were always below plastic limit.

(c) Water contents tended to vary directly with the water plasticity ratio, which is defined as (MC - PL)/(LL - PL) for both base courses and subgrades.

where MC = moisture content
PL = plastic limit
LL = liquid limit

(d) The degree of saturation did not correlate with annual rainfall, but did appear to be related to plasticity as reported by Kersten (1944).

Wesley and Rengga (1972) found that in Indonesia an equilibrium moisture regime is established in the subgrade and that in general it is close to the plastic limit. It did not appear to vary with season.

The Road Research Laboratory in Britain (Coleman, 1949; Croney and Coleman, 1948) adopted the following theoretical approach in simulating water movement. The laboratory assumed a soil having an impermeable surfacing of infinite extent. This reduced the problem to a vertical dimension only. It was also assumed that the soil was at a constant temperature and that the water table position was steady. Under these conditions the pore water pressure at a point x mm above the water table is less than atmospheric pressure by an amount equal to the pressure exerted by x mm of water. Since a pore water pressure value of zero generally corresponds to atmospheric pressure, values above the water table are generally negative.

Thus if the depth of the water table is known, the pore pressure distribution can be calculated. If an overburden pressure exists, the fraction of pressure effective in altering the water pressure in the soil can be found by loading or shrinkage tests. The pore water pressure, u, is then given by:

$$u = \alpha P + \phi_m \quad \text{(3.1)}$$

where ϕ_m = soil suction (negative)

αP = fraction of overburden pressure (P) effective in altering water pressures in soil (positive)

The pore water pressure u is generally found by measurement using tensiometers or other equipment. If u can be measured and αP determined, then the suction ϕ_m can be calculated by the following equation:

$$\phi_m = u - \alpha P \quad \text{(3.2)}$$

Further, if the appropriate relationship between suction and moisture content is known, values of suction can be converted to moisture content and an equilibrium distribution of moisture content with depth can be established.

From experimental work done by Black et al (1959) the following conclusions were drawn:

(a) Seasonal variations under grassed areas were much greater than under bare areas, suggesting that transpiration from vegetation affects soil moisture content much more than evaporation.

(b) The theoretically derived moisture content in the soil was generally ± 2 per cent moisture off the measured moisture content. (Level of confidence not known.)

(c) Under impervious pavements the moisture distribution with depth was largely determined by the position of the water table. A water table fluctuating near the surface resulted in comparatively large fluctuations of mositure content, while reasonable equilibrium conditions were attained where the water table was at a greater depth (deeper than a minimum of 2 m, but depends on material type).

(d) "Edge" effects gave appreciable moisture content changes for a horizontal distance of 0,6 to 0,9 m under the pavement from the pavement edge.

Russam and Coleman (1961) have shown that under English conditions, with water tables not more than 3 - 5 m below the soil surface, the sealed pavement can be considered to be impermeable and of infinite width. Under these conditions the soil moisture suctions are in static equilibrium with the water table. In Trinidad Russam (1958) showed that the theoretical method derived from and varified for conditions in Great Britain, also holds for an approximately uniform temperature (25 °C) and high rainfall (1500 - 3 000 mm). Edge effects and seasonal variation were not evaluated in this investigation.

Russam (1965), in a review of the work done by a study group on moisture conditions in pavements in tropical areas, recommended that:

(a) the ultimate moisture condition of the subgrade can be estimated in terms of soil suction for 3 categories of subgrades: Category (a) comprises subgrades in areas where the water table is within 6 metres of the surface and the suction depends only on the position of the water table. Category (b) comprises subgrades where the water table is deeper than 6 m but the seasonal rainfall exceeds 250 mm. The suction may be deduced from the Thornthwaite's Index and the plasticity index in this case. Category (c) comprises subgrades where the water table is deeper than 6 m but the seasonal rainfall is less than 250 mm. The suction may be deduced from the atmospheric relative humidity in this case.

(b) Good drainage should be provided.

(c) Care must be taken that water is not trapped in the pavement during construction.

(d) Where conditions of deep water table and good standards of construction can be met, a conservative estimate of the critical subgrade strength can be obtained by testing at the optimum moisture content and maximum dry density determined by the British Standard Compaction Test.

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(d) Where conditions of deep water table and good standards of construction can be met, a conservative estimate of the critical subgrade strength can be obtained by testing at the optimum moisture content and maximum dry density determined by the British Standard Compaction Test.

Th' ɔad Research Laboratory extended its investigations to areas having widely different climates to determine whether their method of predicting the ultimate moisture distribution was still applicable. Experimental sites in Aden, Egypt, Hong Kong, Singapore, Iraq, Rhodesia and Sudan showed that the theoretical method was applicable where a water table was present close to the surface. The limiting depth for the method was, where a water table existed, about 9 m in clayey sand and 3 m in sand. It was also found that in an arid climate and in the absence of a water table the moisture content of the soil was controlled by the atmospheric humidity. The movement of water in the vapour phase appeared not to be a factor in controlling subgrade moisture contents except where a relatively large temperature fluctuation (e.g. a variation in soil temperature of 6 °C occurred in 15 days) caused a thermal gradient to be established in the soil.

Further investigations were performed in Kenya, Tanzania and Uganda by Williams et al (1962). The purpose of these investigations was to determine the moisture conditions in subgrade soils in tropical climates. It was found that where the water table is at depths greater than 1 m, in sands, and 6 m, in heavy clays, below the road surface, the main factors affecting the subgrade moisture conditions are climate and soil permeability. The study also showed that the moisture content at subgrade level is in general at, or drier than, plastic limit and less than the optimum moisture content in the British Standard compaction test. Scala (1967), after a detailed investigation of the influence of climate on the ultimate moisture conditions in subgrades in Australia, concluded that climate is a very important factor determining the ultimate suction in the subgrade. He presented a relationship between climate and the equilibrium suction.

In Kenya, Russam and Dagg (1965) found that, for a permeable shoulder, the rate of infiltration into the shoulder could be reduced by increasing the shoulder crossfall. Kikuyu grass dried the road verge to a depth of 3 m but the moisture content of the soil beneath the pavement remained virtually unchanged. An investigation to determine the seasonal changes of moisture in the road subgrade was carried out in Rhodesia by the Ministry of Roads and Road Traffic (1963). The observations showed that the effect of the sealed road pavement is to reduce evaporation losses and rainfall effects so that for a distance of 0,3 m from the edge of the seal, the seasonal variation in the subgrade has been reduced to less than a half of the fluctuations occurring on the verge. Uppal (1965) reported that in black cotton soil subgrades in India the moisture content varied from 6 per cent to 23 per cent during the year under the shoulders, whereas the variation under the pavement was from 13 per cent to 16 per cent. His survey also showed that the phenomenon of "equilibrium moisture content" as found in England under sealed surfaces, was not observed in India. He inferred, however, that "for the prevailing rainfall intensities (or water table conditions) in India, the design of the road pavement need not be carried out at complete saturation for all the places in India". He also reported the following interesting observations:

(a) The moisture content in the uncovered soil of the top 1 m varies between the shrinkage limit and plastic limit with season.

(b) There is a moisture gradient with depth under a sealed and unsealed surface to a depth of 2,5 m, where a stable moisture content is formed.

(c) An uncovered shoulder has no significant effect on the subgrade further than 1 m from the edge of the pavement.

Wallace (1978) pointed out that the ideal situation would be for the design engineer to be able to predict the exact extent of the wetting up of a particular pavement and subgrade, but this is virtually impossible. He showed that the shoulder permeability is the critical factor determining water ingress through the shoulders. He also found that the potential benefit of good drainage is greater in a wet climate.

Very little work has been done on the moisture conditions in other pavement layers, but McInnes (1972) reported that the moisture content in the base of relatively lightly trafficked roads (< 200 v.p.d.) very seldom reach the Modified AASHTO optimum moisture content. The ratio of field moisture content to Mod. AASHTO optimum moisture content ranged from 0,32 to 1,04 with an average ratio of 0,66. He also found that a variation of 2 percentage points in the base moisture content seemed abnormal.

As can be seen, many conflicting conclusions have been drawn through the years and an attempt will be made forthwith to summarize the results.

3.3 General conclusions

3.3.1 Influence of surface drainage

Redus (1957) could only conclude that surface drainage is a contributing factor to pavement failure. The U.S. Corps of Engineers (1955) reached a similar conclusion, stating that good drainage will not ensure satisfactory performance. Marks and Haliburton (1969) reported that good surface drainage, which removes run-off quickly and completely, reduces moisture variations from infiltration through the shoulders.

It should be pointed out here that it is very difficult to isolate the influence of drainage on the subgrade moisture regime, because the amount of water reaching the subgrade is dependent on the permeability of all the structural pavement layers and not only the surfacing. It is obviously difficult to measure these permeabilities. Scala (1967) reported that the assessment of drainage by a team of competent engineers was extremely difficult and only major changes in drainage conditions could be detected as being significant. It has, however, been shown (Cedergren, 1974) that bad surface drainage will invariably adversely affect the moisture conditions in the structural pavement layers above the subgrade. It has also been shown by various workers (Cedergren, 1974; Chu and Humphries, 1974; Morris, 1976) that proper subsurface drainage will improve the moisture conditions throughout the pavement structure including the subgrade. What is clear is that no ponding should be allowed and that surface water should be removed quickly.

3.3.2 Influence of road condition

The U.S. Corps of Enigneers (1955) found that moisture contents measured adjacent to cracks were about the same as those measured at normal locations, except in the base adjacent to the cracks in the low rainfall zones, where moisture was significantly lower than in the other locations. This was observed for the base, sub-base and subgrade and all climatic regions were included. They also could not relate the voids (permeability) in the bituminous surfacing to the above observation except that the percentage voids should be less than 7 per cent. It was also reported that after a crack had opened up, the moisture content generally increased somewhat immediately after; subsequently it might stay constant, dry out or wet up. There was no trend for moisture to increase or decrease with distance from crack. Russam (1962) reported that an impervious surfacing reduces the variation in moisture content in a subgrade and a more stable value is approached. The Rhodesians Mitchell and Venter (1974) also reported a removal of moisture content extremes in the upper pavement layers by resealing the surface.

3.3.3 Influence of time

Redus (1957) indicated that the moisture content in subgrades stabilizes after two years. Guinee and Thomas (1955) reported that the variations become smaller with depth. Chu and Humphries (1971) reported that subgrade moisture content varies with time by up to 6 per cent. Many researchers (Marks and Haliburton, 1969; Hicks, 1948) reported slight variations with season and the highest moisture content in the dry season. Kersten (1945) reported only slight changes in moisture content over periods of up to 5 years. Russam (1962) also reported a tendency for subgrade moisture contents to reach a stable value. Marks and Haliburton (1969), on the other hand, reported no stable conditions on the sites they sampled except where the water table was consistently high. It is possible that they sampled sites which had not reached equilibrium moisture content, because there is overwhelming evidence (Aitchison and Richards, 1965; Richards, 1967; Black et al, 1959; Russam, 1962; O'Reilly et al 1968; Scala, 1962) that the soil under covered areas (except near the edges) tends towards a stable moisture distribution.

3.3.4 The moisture variation across pavement width

Russam (1962) reported that, after equilibrium had been reached, the subgrade under the pavement became wetter with increasing distance from the boundary. Even in arid regions this observation seems to hold, although the initial increase in moisture content is higher in these cases due to surface run-off. This does not seem to be true in all cases, as the U.S. Corps of Engineers (1955) reported some cases where the opposite trend was true. They also found some sites where the moisture content showed no trend across the pavement. It is believed by the author that the transverse gradient is [illegible]nt on the method of wetting up and on the time of year measure[illegible]en, because the shoulders generally don't reach equilibrium con[illegible]f it wets up from the atmosphere (when the water table is deep) the edges should be wettest. If it wets up from the groundwater (water table is shallow) the centre should be wettest. This is assuming that the surfacing of the pavement is impervious.

3.3.5 Influence of the water table and of climate

Russam and Coleman (1961) and Aitchisor and Richards (1965) concluded from their studies of climate and subgrade moisture that where the water table is near the surface, it becomes the dominating factor controlling the moisture distribution under sealed surfaces. Where the water table is deep, the main factors affecting the moisture content are rainfall and evapotranspiration and soil type. This is borne out by the general agreement between Thornthwaite's moisture index and the soil suction at equilibrium. Although this seems logical, the U.S. Army Corps of Engineers (1955) have on the other hand reported that there exists no relationship between mean annual rainfall and the moisture content in the base, subbase and top of the subgrade. In a wet climate, the moisture content 500 mm into the subgrade tends to be higher though. Both high and low rainfall areas were considered, but the depth to the water table was not measured.

3.3.6 Depth to stabilized moisture content

Uppal (1965) reported that a stable moisture content was attained in the veld at ± 3 - 4 m depth. Although the climate only influenced the top 1 m, a fluctuating water table caused the deeper variations. Under a covered area, equilibrium is reached at ± 1,5 m, but in some cases only below 2,5 m. Aitchison and Richards (1965) reported seasonal variations in the top 1 - 3 m in the open field throughout Australia. They also reported stable moisture conditions directly under the seal. The U.S. Army Corps of Engineers (1955) also reported stable moisture conditions in the base, sub-base and subgrade.

3.3.7 Influence of temperature

Uppal (1965) found that there existed no relationship between temperature gradient and moisture content in the open veld. Marks and Haliburton (1969) reported a strong temperature dependence of moisture content under a paved area. NITRR (Chapter 6) also found that there exists a strong relationship between temperature and moisture content under pavements, especially in the upper pavement layers where the temperature gradient may be steep.

3.3.8 Influence of shoulders

Uppal (1965) reported that the shoulders had no effect on the moisture content of a clay subgrade in any season, from a distance of 75 cm from the edge. This distance is reported to be dependent on material type. The lateral influence of the shoulder is felt deeper in a sandy subgrade than in a clayey one. Russam and Dagg (1965) reported that the slope of the shoulder had considerable influence on the moisture infiltration for permeable soils. Steeper slopes reduced infiltration. Kikuyu grass dried the shoulders to a depth of 3 m, but also caused more infiltration, causing larger moisture changes. Polythene membranes stopped light rain from infiltrating, but were ineffective against prolonged rain. They caused generally wetter conditions because they stopped evaporation.

3.3.9 The influence of soil types

Several researchers have reported how moisture content varies with soil type and the phenomenon is well known. Generally moisture content is found to vary directly in relation to the percentage finer than 0,075 mm, the plasticity and optimum moisture content. The U.S. Army Corps of Engineers (1955) found that the moisture content of subgrades sometimes exceeded the plastic limit but never the liquid limit. Marks and Haliburton (1969) reported that soil type had no effect on subgrade moisture variations beneath Oklahoma highway pavements, but this conclusion is unreliable due to insufficient sampling.

3.3.10 Degree of saturation

The U.S. Army Corps of Engineers (1955) found that base courses and subgrades with plasticity indices of about 5 or more, showed degrees of saturation of 85 per cent or more for most of the time, irrespective of rainfall. They also reported that the degree of saturation for both bases and subgrades varied directly with plasticity. It also varied with percentage finer that 0,075 mm in the case of subgrades. Van Der Merwe (1967) estimates that 10 per cent of the random field moisture content determinations will equal the soaked value. Several research workers (Russam, 1962; Uppal, 1965) reported that no evidence could be found to substantiate the assumption that the top of the subgrade becomes saturated under normal circumstances. It was found by Mitchell and Venter (1974), Mitchell and Northrop (1973) and Van Der Merwe (1967), however, that in some cases the whole pavement may become saturated prior to failure. Richards et al (1970) found in Queensland, Australia, that the equilibrium moisture content for their clayey subgrades is "considerably drier than optimum moisture content for standard compaction". They found the equilibrium suction to be 2400 kPa (pF 4,38), which is 6 per cent lower than the optimum moisture content.

3.3.11 Measures that could be taken to ensure stable moisture conditions

(a) Design surface drainage facilities properly so that no ponding occurs on the pavement, the shoulders or in the side ditches. The water should flow away from the pavement.

(b) Maintain the seal in an impervious condition. Although a cracked pavement does not cause generally wetter conditions, it causes larger moisture variations, and the pavement layers deteriorate quickly during wet periods.

(c) Avoid grass on permeable shoulders, because it causes greater infiltration.

3.4 Review of methods for estimating moisture content of pavement layers

Since the early stages of the development of the CBR test and design method, it has been the intention to test the material at the worst possible moisture condition likely to occur in the field. Since no reliable information about moisture conditions was available, saturation was assumed. As various countries started moisture investigations, attempts were made to determine at what moisture content the test should be carried out. The Road Research Laboratory (1952) suggested that the pavement be designed for the particular value of moisture content called the "equilibrium moisture content" and not full saturation. A great deal of research work on the movement of moisture in different types of soil has been carried out in various countries and this has led to the modification of the traditional test method in some countries (Croney et al, 1958; Croney and Coleman, 1947; Black et al, 1959; Russam, 1958;

Croney et al, 1952; Schofield, 1935; Aitchison, 1951, 1956; Winterkorn, 1953). Some modifications to the traditional method will be described here.

3.4.1 General

The Organisation for Economic Co-operation and Development (OECD, 1973) has indicated that the methods for predicting moisture movement and moisture equilibria in pavement systems can be placed into two categories as follows:

(a) Empirical methods, based on experience and field studies.

(b) Theoretical methods, based on the equations governing moisture conditions in porous materials.

A third category, namely rational models, is introduced here for completeness' sake. This category is based on a rational approach to the problem and is not based solely on the flow equations or the empirical relationships.

A comprehensive list of abbreviations used in this dissertation is given in Appendix A.

3.4.2 Empirical methods

Because of the complexity of the thermodynamics governing moisture movement in pavement systems and the inaccuracy with which essential parameters can be determined, a substantial amount of work has been done in attempting to relate moisture content to geotechnical characteristics such as plastic limit (PL), liquid limit (LL), optimum moisture content (OMC), percentage finer than 0,075 mm (% - 0,075). Some of the most successful relationships thus developed will be discussed here.

3.4.2.1 The Thornthwaite moisture index method (Russam, 1962; Thornthwaite, 1948)

When the water table is deep, the moisture content is determined by the climate and can be estimated by the Thornthwaite method. Precipitation and evapotranspiration are dominan factors in this case. Thi method is based on the following equilibrium equation:

$$\Delta = P - E - D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.3)$$

where Δ = change in the height of water stored
P = precipitation
E = evapotranspiration
D = drainage

and these are usually quantified in terms of centimetres or millimetres. As E is difficult to assess, Ep is used and it is defined as the potential evapotranspiration. Thornthwaite related Ep to weather information only by the express[illegible]

$$E_p = \text{[illegible]} \left(\frac{\text{[illegible]}0t}{I}\right)^a \text{ cm} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.4)$$

where t = mean monthly air temperature
I = annual total of $(t/5)^{1,514}$
$a = 6,75 \times 10^{-7} I^3 - 7,7 \times 10^{-5} I^2 + 1,792 \times 10^{-2} I + 0,49239$

This gives the monthly unadjusted potential evapotranspiration for a month of 30 days with 12 hours daylight per day. A correction factor for daylight can be deduced from the latitude of the station.

The precipitation P can be measured by normal conventional methods. The drainage D can be estimated by taking the natural drainage conditions, the soil profile, etc. into account (see Russam and Coleman, 1961 for details).

Thornthwaite also defined an index to assess the overall availability of moisture during the year.

$$\text{Thornthwaite's moisture index} = \frac{100D - 60d}{Ep} \quad \ldots\ldots\ldots\ldots \quad (3.5)$$

where Ep and D have the meanings previously ascribed, and d is the soil moisture deficit (given approximately by d = Ep - p - 4). This method, 3.4.2.1 The Thornthwaite moisture index method (Russam, 1962; Thornthwaite, 1948)

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$$E_p = 4{,}06 \, \frac{(10t)^a}{I} \text{ cm} \quad \ldots\ldots \quad (3.4)$$

where t = mean monthly *air temperature*

I = annual total of $(t/5)^{1{,}514}$

$a = 6{,}75 \times 10^{-7} I^3 - 7{,}7 \times 10^{-5} I^2 + 1{,}792 \times 10^{-2} I + 0{,}49239$

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where Ep and D have the meanings previously ascribed, and d is the soil moisture deficit (given approximately by d = Ep - p - 4). This method, using the Thornthwaite moisture index, therefore allows the assessment of climate using temperature, duration of possible sunshine and rainfall. The Thornthwaite moisture index gives rise to 3 different climatic categories: Wet ($d = 0$, always), Dry ($D = 0$, always) and Intermediate ($d = 0$ for some months and $D = 0$ for other months). In the wet regions water drains from the soil almost continuously and little change of moisture content or strength occurs with season. Moisture conditions under the pavement are similar to those in the exposed soil. The dry category corresponds to desert or semi-desert conditions and seasonal changes of moisture content are small. In general the moisture content of the soil is governed by the average humidity of the atmosphere. The equilibrium soil moisture suction can be estimated from the moisture index through the use of the relationship between soil suction and the index, as shown in Figure 3.1. The next step is to determine the dry density versus moisture content versus suction relationship for each soil type along the proposed road. From this relationship, the predicted equilibrium suction and the specified compaction required, the equilibrium moisture content can be determined.

3.4.2.2 Swanberg and Hansen (OECD, 1973; Swanberg and Hansen, 1946)

These investigators found that the equilibrium moisture contents (EMC) of highway subgrades in Minnesota could be estimated in terms of the plastic limit (PL) as follows:

$$EMC \approx 1{,}16(PL) \; [illegible] \; (\%) \qquad \ldots \qquad (3.6)$$

The subgrades investigated were mainly clayey silt soils with plastic limits from 15 to 30 per cent. The density was 90 to 105 per cent of the maximum density by the Mod. Proctor Test, AASHTO T99-38. The moisture contents measured in the spring were about 0,8 per cent higher than those measured in summer, in this summer rainfall area.

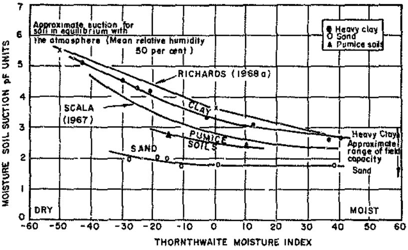

FIGURE 3.1

VARIATION OF SOIL SUCTION OF ROAD SUBGRADE WITH THORNTHWAITE MOISTURE INDEX FOR SITES WHERE NO WATER TABLE WAS LOCATED (SCALA, 1967; RUSSAM AND COLEMAN, 1961; RICHARDS, 1968 a)

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3.4.2.3 Haliburton (1971)

Haliburton (1971) found that an equilibrium moisture content of approximately 1,1 to 1,3 times the subgrade plastic limit is usually reached.

3.4.2.4 Livneh and Ishai (1975)

They proposed that the equilibrium moisture content (EMC) may be estimated as follows:

$$\frac{EMC}{PL} = 0,86 + 0,14(PL) \ldots\ldots (3.7)$$

where $\frac{EMC}{PL}$ ranges between 0,9 and 1,5 and PL = plastic limit.

3.4.2.5 Madrid Transport and Soil Mechanics Laboratory (OECD, 197'

This investigation reported the following relationships:

0,87 (PL) < EMC < 1,11 (PL) standard deviation 0,11 - 0,42 ... (3.10)

0,33 (LL) < EMC < 0,49 (LL) standard deviation 0,07 - 0,17 ... (3.9)

0,33 (% - 0,075) < EMC < 0,60 (% - 0,075) standard deviation 0,08 - 0,29 .... (3.10)

where EMC = equilibrium moisture content

LL = liquid limit

(%-0,075) = percentage material finer than 0,075 mm.

None of these relations should be used except as for the determination of a first estimate, because of the lack of accuracy. It was also found that the standard deviations did not diminish with increase in depth.

3.4.2.6 United States Navy (OECD, 1973; U S Navy, 1953)

The U.S. Navy investigated the sandy and clayey subgrades of 70 airports where the groundwater table was more than 60 cm below the surface. The following relationship emerged:

$$EMC \leq PL + 2\% \quad \text{(3.11)}$$

The major conclusion was that the moisture content hardly ever exceeded the PL by more than 2 per cent.

3.4.2.7 Kersten (1944, 1945)

Kersten investigated the subgrade moisture contents of airport pavements in seven U.S. states with damp to desert climates. The moisture contents were observed in the top 30 cm of the subgrade soil. He presented the following equation:

$$0,8\ (PL) < EMC < 1,2\ (PL) \quad \text{(3.12)}$$

3.4.2.8 Wooltorton (1958)

The Highway Research Board presented the following equation based on a large amount of published experimental data:

$$EMC = 1,17\ PL - 4 \quad \text{(3.13)}$$

3.4.2.9 Organisation of Economic Co-operation and Development (1973)

The OECD proposed that moisture content could be estimated as follows:

$$EMC = 0,9\ \gamma_w \left(\frac{1}{\gamma_d} - \frac{1}{\gamma_s} \right) \quad \text{(3.14)}$$

or $EMC \approx 0,9\ S_r$ for clayey soils

where γ_w = wet density

γ_d = dry density

γ_s = saturated density

S_r = degree of saturation

3.4.2.10 Road Research Laboratory (1977)

The RRL suggested the following empirical rule in Road Note 31:

$$\frac{\text{Field moisture content}}{PL} = \text{constant} \qquad (3.15)$$

3.4.2.11 Brodie (1970)

In 1969 Brodie analysed 330 samples statistically in order to be able to predict the performance of a pavement from various contributing factors. He also performed an analysis to establish a prediction model for moisture content. The following model was proposed:

$$EMC = 0,825\ (OMC) - 2,7 \qquad (3.16)$$

This equation has a multiple correlation coefficient, r, of 0,78 and optimum moisture content in this case is for British and not Proctor compaction. No improvement resulted from the inclusion of a climatic index like mean annual rainfall. This equation was developed from all material types, from all pavement layers, finer than 19 mm (3/4"). When done only for subgrade soils, no change was effected. The inclusion of grading analyses also produced no improvement.

3.4.2.12 Gawith (1948); Croney <u>et al</u> (1953)

The Country Road Board, Victoria, Australia, undertook a statistical analysis of more than 100 road investigations in an attempt to relate the field moisture content to simple soil tests. The equations they developed are as follows:

$$EMC = 0{,}5278(PL) \quad 0{,}1183(LL) + 0{,}2359(\% - 0{,}075) - 0{,}0006864(\% - 0{,}075)^2 + 0{,}1396(MAR) - 14{,}29 \quad \ldots\ldots \quad (3.17)$$

$$EMC = 0{,}6776(PL) + 0{,}07875(LL) + 0{,}2073(\% - 0{,}075) - 0{,}000498(\% - 0{,}075)^2 - 10{,}681 \quad \ldots\ldots \quad (3.18)$$

3.4.2.13 Arulunandan (1957)

Aralunandan proposed that the equilibrium moisture content could be predicted by the following equation:

$$EMC = \frac{PL + (\% - 0{,}075)}{5} \quad \ldots\ldots \quad (3.19)$$

3.4.2.14 Van Der Merwe (1967)

Van Der Merwe proposed the following relationship:

$$EMC = \frac{PL}{(0{,}001(MDD) + 0{,}17)} \quad \ldots\ldots \quad (3.20)$$

Figure 3.2 shows a comparison of the various methods involving plastic limit.

3.4.3 <u>Comments on empirical formulae</u>

The OECD concludes that the available empirical formulae for the

3.4.2.12 Gawith (1948); Croney et al (1953)

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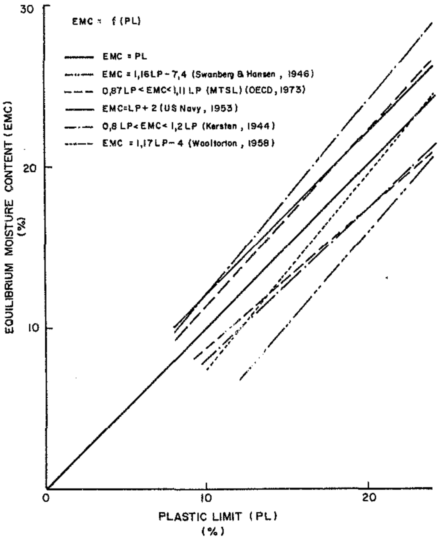

FIGURE 3.2

EMPIRICAL RELATIONSHIPS BETWEEN MOISTURE CONTENT AND PLASTIC LIMIT (OECD, 1973)

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determination of moisture contents cannot really be described as methods of prediction. They point out that the results of the various investigations are scattered around the line representing EMC = PL. Figure 3.2 shows that a general treatment of all the data collected leads to a zone extending approximately 4 percentage points on either side of the line of eqaulity.

This is a fairly wide range in which the variation of the physical properties may be quite large. For example, the CBR could vary by a factor higher than 2.

The OECD listed the following causes for the considerable dispersion of the empirical formulae:

(a) The water contents observed have not been correlated with the more fundamental characteristics of t ure of the soils such as the specific surface area and the ogical composition.

(b) The compaction of the soil has not been measured, with the exception of those investigations which give some consideration to the degree of saturation. It is obvious that the quantity of water a soil can absorb in given circumstances varies with compaction.

(c) The degree of waterproofing of pavements has not been measured; in general the t of pavement has not been mentioned.

(d) The edge effect have been disregarded in most studies.

(e) The absence or presence of a groundwater table and its depth generally does not appear to have been taken into consideration. This is also the case with the water content of adjacent soil masses.

(f) The effect of climatic factors has not been disclosed.

The conclusion that the empirical formulae will generally give rather poor solutions should not mean that considera local interest may not be attached to the investigations. It is possible that certain engineering design decisions could be made on the bases of these predictions, provided it is in the region where the investigations were carried out or under similar conditions. These methods of prediction are naturally also applicable in cases where the error in prediction is known and acceptable to the specific situation

3.4.4 Rational methods

The British Road Research Laboratory has produced a rational method of predicting moisture movement based partly on theoretical considerations and partly on field observations.

3.4.4.1 The British Road Research Laboratory Method (Russam, 1962; Croney and Coleman, 1948; Russam and Coleman (1961).

After extensive research (Black et al, 1959; Russam, 1962; Russam and Coleman (1961), the British Road Research Laboratory produced a rational method of estimating the design moisture content (Road Note 31). The method is based on the assumptions that:

(a) the temperature conditions in the subgrade are constant, uniform and above freezing;

(b) the subgrade cannot wet up by infiltration through the pavement or by migration from adjacent soil masses with a higher (more positive) water potential, nor can it dry out by evaporation or migration to adjacent soil masses having a lower (more negative) water potential.

These assumptions are closely approximated when the subgrade is covered with a pavement whose permeability is at least ten times lower than that of the subgrade. The pavement edges are also affected by moisture transfer, but the subgrade under the centre line is unaffected by this. They found that the subgrade moisture conditions under impermeable pavements can be classified under three main categories:

(a) Subgrades where the water table is sufficiently close to the surface to control the moisture conditions in the subgrade. The type of subgrade material will determine at what depth the water table becomes the dominant factor influencing subgrade moisture conditions. In non-plastic soils the water table becomes dominant when it rises to within 1 m of the surface. In sandy clays (PI $\leqslant$ 20) the water table will dominate when it is in the top 3 m from the surface and for heavy clays (PI $\geqslant$ 40) the water table is dominant if it is in the first 7 m from the surface. In addition to areas where the water table is maintained by rainfall and impeded drainage, this category also includes coastal strips and flood plains of rivers, where the water table is maintained by sea, lake or river.

(b) Subgrades where the water table is deep but where the rainfall is sufficient to produce significant seasonal changes in the subgrade moisture conditions. In these cases the rainfall is greater than 250 mm and it is often seasonal.

(c) Subgrades where the water table is deep and where the climate is arid throughout the year, with a mean annual rainfall less than 250 mm.

Different methods of estimating the moisture content in each of the three categories given above were developed.

Category (a)

For this category the water table is sufficiently close to the surface to be the dominant factor controlling the matric suction. The level of the water table varies with season and the highest level is used as the "equilibrium" value (Russam, 1962). In this case the suction at a point is related to the difference in elevation between that point and the water table. The suction profile is given by:

$$\phi_m = -z \quad \ldots\ldots \quad (3.21)$$

where ϕ_m = is the matric suction at point z m above the water table.

In this case the matric suction is identical to the negative pore water pressure because there is no change in overburden pressure. Allowance for any change in overburden pressure should be made and can be simply estimated from the relation

$$\Delta u = \alpha \text{ (change in overburden pressure)} \quad \ldots\ldots \quad (3.22)$$

where α = 0,03 x (Plastic index) (Croney, 1952).

The distribution of overburden stress between the water pressure and the soil intergranular pressure has been studied by Bishop (1960) and Skempton (1960). They showed that, for non-saturated non-cohesive soils, the factor is generally near zero, while for cohesive soils α increases with the degree of saturation and reaches the value 1 when saturation is complete.

The suction is related to the pore water pressure as follows:

$$u = \phi_m + \alpha P \quad \text{.......} \quad (3.1)$$

where u = pore water pressure

ϕ_m = matric suction

P = overburden pressure

α = compressibility factor

A change in overburden pressure may thus affect the pore water pressure but not the suction. If the change in overburden pressure causes a density change, then the moisture content will change but the suction may remain constant. The characteristic moisture content-suction relationship will also change because it has to be carried out on a sample representative of density conditions. The value of ϕ_m in equation 3.1 will always be negative in practice and that of P positive or zero, thus if αP is numerically greater than ϕ_m the pore pressure will be positive and if the reverse is the case it will be negative. The matric suction is thus the pore water pressure when the sample is relieved of all external stresses.

In order to estimate the ultimate moisture content, it is necessary to know the relationship between suction and moisture content for that specific material. This relationship can be obtained in the laboratory by determining for successive specific values of the pore water pressure in a soil free from all external stress, the corresponding moisture content where no further movement of water occurs. Hysteresis has a strong effect on this relationship and care should be exercised to represent field conditions in the testing program, i.e. either wetting or drying.

The application of the rational method is shown in Figure 3.3.

(a) Calculate the values of the overburden pressure P and product αP at the desired depths.

(b) The estimated level of the groundwater table is determined and for the same depths as in (a) the values of the equilibrium pressures are determined by equation (3.21) and those for the equilibrium suction equation (3.1).

(c) The equilibrium moisture content corresponding to the equilibrium suctions determined in (b) above are read off the soil moisture characteristic curve for the expected soil. The curve is characteristic for one soil type under certain physical conditions and must be

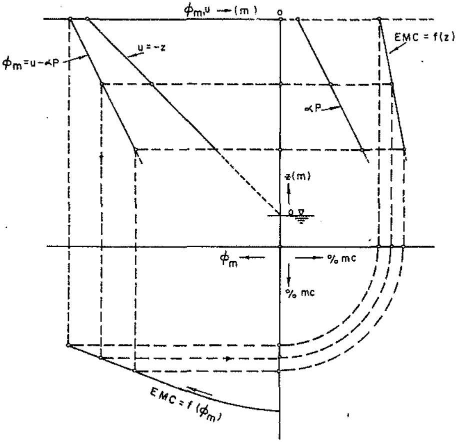

FIGURE 3.3

ILLUSTRATION OF APPLICATION OF THE R.R.L METHOD WHEN A WATER TABLE IS PRESENT AT A SHALLOW DEPTH (OECD, 1973)

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determined in a laboratory.

(d) For the different depths chosen, the corresponding points of the profile of equilibrium moisture contents are thus obtained.

This method has very successfully been applied to heavy clays in Britain. Croney et al (1953) simply assumed an equilibrium suction of pF 2,5 for all of Britain and from this assumption and the soil moisture characteristic curve of each soil type sampled, they calculated the equilibrium moisture content. Very good correlations with actual moisture contents were observed over a very wide range of moisture contents. Care must, however, be exercised to determine the applicability of this method by examining the groundwater level, the permeability of the pavement structure and the climatic factors.

As an alternative, if laboratory equipment is not available, the ultimate moisture content can be determined by measuring the moisture content in subgrades below existing pavements in similar situations at the time of the year when the water table is at its higher level (just after the rainy season). These pavements should be more than 3 m wide and more than 2 years old. Allowance can be made for different soil types by virtue of the fact that the ratio of subgrade moisture content to plastic limit is the same for different subgrade soils when the water table and climatic conditions are similar. (The ratio between the adjusted liquid limit and the subgrade moisture content is significantly better for southern African conditions, i.e. $LLT = (LL)^{0,7}(\% - 0,425)^{0,3}$).

Where there are no existing pavements in similar situations available for comparison, an estimate of the minimum CBR can be obtained from a table prepared for soils compacted to not less than 95 per cent of the maximum dry density attainable in the British Standard Compaction Test, 2,5 kg rammer method. These values are minimum and the CBR should be measured in the laboratory wherever possible at the appropriate moisture content. These values should not be used with any design method but with the design charts in Road Note 31.

Category (b)

In this case the subgrade moisture condition under an impermeable pavement will depend on the balance between the water entering the subgrade through the shoulders during wet weather and the moisture leaving the ground by evapotranspiration when the weather is dry (O'Reilly et al, 1968). The ultimate subgrade moisture condition can be assumed to be the optimum moisture content given by the BS Compaction test (2,5 kg rammer method. (This is approximately equivalent to the Standard Proctor effort.)

Category (c)

The subgrade moisture content for this category where the climate is arid throughout the year (annual rainfall 250 mm or less) will be virtually the same as that of the uncovered soil at the same depth. This moisture content should be used for design purposes.

3.4.4.2 Richards (1968b)

Drawing from Australian investigations (Aitch... ...d Richards, 1965; Richards, 1967; Scala, 1962) and similar investigation elsewhere (Black et al, 1959; Russam, 1962; O'Reilly et al, 1968), Richards (1968b) proposed the following prediction technique for equilibrium suction:

(a) Shallow water tables: (7 m in clays; 3 m in sandy clays and silts and 1 m in sands)

Where the water table is shallow, then according to Croney (1963) the suction profile will be in hydrostatic equilibrium with the water table irrespective of the climate. Hence,

$$(h_m)z = z_w - z \quad \ldots\ldots \quad (3.23)$$

where $(h_m)z$ = matric suction at depth z in meters of water

z_w = depth of water table from surface

(b) Deep water table

Where the water table is deep, the moisture conditions in the stable zone are controlled by a moisture balance between rainfall and evapotranspiration. Various relationships between climate and equilibrium soil suction have been obtained (Aitchison and Richards, 1965; Russam and Coleman, 1961; Thornthwaite, 1948). Although factors like through the shoulders during wet weather and the moisture leaving the ground by evapotranspiration when the weather is dry (O'Reilly et al, 1968). The ultimate subgrade moisture condition can be assumed to be the optimum moisture content given by the BS Compaction test (2,5 kg rammer method. (This is approximately equivalent to the Standard Proctor effort.)

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(c) Extrapolation from deep profiles

(i) Areas where no permanent surface desiccation exists.

Aitchison and Richards (1965) have shown that for these areas the suction profile under a covered area is identical to the suction profile under natural surface cover, below the depth of seasonal variation. This seasonal variations in the open field occur to depths of less than a metre to approximately 3 m throughout Australia. It was concluded that a structure of 7 - 10 m width appears to have negligible effect on moisture conditions below the depth of seasonal variation. The equilibrium suction profile beneath the structure can therefore be predicted by extrapolation of the near equilibrium profile below the depth of seasonal variation. It is usually assumed that the same suction existing below the depth of seasonal variation *occurs at the* surface.

(ii) Where permanent surface desiccation occurs.

Russam and Coleman (1961) and Aitchison and Richards (1965) showed that for such dry areas, the subgrade moisture conditions may be determined from the mean atmospheric humidity. Where deep desiccation, relative to the width of the structure ($\pm$ 3 m) occurs, the moisture conditions are unaffected by the structure and the equilibrium suction can be predicted to be that measured at 3 m prior to construction.

3.4.4.3 *The Regional Basis Method* (NAASRA, 1974)

Maps showing zones of uniform equilibrium suction need to be prepared for this method. With this information the procedure is as follows:

(a) Ensure that local factors which are likely to affect the equilibrium suction are absent. These include perched water tables, inundation by flood water irrigation, etc.

(b) From the relevant area, read the predicted equilibrium soil suction from the map of equilibrium suctions.

(c) For each soil type along the proposed alignment, determine the dry density vs. moisture content vs. suction relationship.

(d) From this relationship, the predicted equilibrium suction for the soil and the specified standard compaction required, determine the moisture content of specimens.

3.4.4.4 Site investigation method (NAASRA, 1974)

This method is conducted as follows:

(a) Delineate the terrain into units of similar physiographic features. This can be done either from aerial photographs or field inspection.

(b) Take undisturbed samples at selected positions in each unit to depths below the zone of seasonal variation.

(c) Carry out laboratory tests to determine the suction profile at each of the sites sampled. Alternatively measure moisture contents on each sample and plot the profile of the ratio of moisture contents to optimum moisture content and plastic limit.

(d) If the suction has been measured, the following procedure should be taken:

Determine the equilibrium suction from the following equation

$$\phi_z = \phi_{zo} + \rho_w(z_o - z) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3.24)$$

where ϕ_z = equilibrium suction beneath covered area at depth z

ϕ_{zo} = suction at depth z_o which is below the zone of seasonal moisture variation

ρ_w = density of water

In areas of high suction where no water table is present, the relative magnitude of the second term is negligible and hence $\phi_z = \phi_{zo}$. In areas where the water table is present, the suction ϕ_z is determined by considering the critical level of the water table. Here $\phi_{zo} = 0$ and $\phi_z = \rho_w(z_o - z)$.

(e) On the basis of the foregoing data on physiographic features and suction determinations, the equilibrium suction along the entire road centre line can be interpreted.

(f) For each soil type along the proposed alignment, the dry density vs. moisture content vs. suction relationship must be determined.

(g) From this relationship, the predicted equilibrium suction for the soil and the specified standard of compaction, the equilibrium moisture content can be determined.

If the moisture contents were measured in (c) above, the following procedure would be followed:

(d_2) On the profile of relative moisture contents extrapolate the relationship occurring below the zone of seasonal variation to determine the equilibrium relative moisture content at subgrade level.

(e_2) Determine the optimum moisture content or plastic limit of the surface material and hence calculate the equilibrium moisture content at subgrade level.

3.4.4.5 Extrapolation from existing roads (NAASRA, 1974)

It is also possible to predict moisture conditions by extrapolation from existing roads as follows:

(a) Select sections of existing road with conditions similar to those which will exist for the road being designed.

(b) Within the section chosen for investigation, select a number of sites for sampling. The longitudinal location of sites will be on a random pattern. The lateral location of sites will normally be in the outside wheelpath unless the cross section of the road differs markedly from that proposed to be used.

(c) Sample the subgrade at the locations selected.

(d) Measure either the suction of the samples or the moisture content and soil parameters like optimum moisture content, plastic limit and liquid limit.

(e) If the suction has been measured, it must be assumed that this will be the equilibrium suction under the proposed pavement and the same procedure outlined before must be applied to arrive at an equilibrium moisture content value from this equilibrium suction.

(f) If the relative moisture content or the moisture content ratio with some soil parameter had been measured, then it must be assumed that this ratio will be the same under the proposed road. The relevant soil parameter must be determined for the proposed road subgrade conditions and from this the equilibrium moisture content can be deduced.

3.4.5 Comments on rational methods

Of all the available moisture prediction techniques the rational methods have won most practical application. The main disadvantage lies in the fact that it is much more laborious to apply the rational methods than the empirical ones. It is also not clear whether a substantially more accurate estimation of the moisture conditions may be made, as most methods include empirical relationships in any case. The British method, for instance, cannot be applied to local conditions without establishing a characteristic suction-moisture content curve or employing some empirical relationship. It is possible that the method is only as accurate as the least accurate step and one may therefore argue that little accuracy increase is achieved if empirical relationships are involved in any case.

It is, however, clear that these methods have found practical applicability in countries like Britain and Australia and therefore should be investigated further to establish applicability to local conditions.

3.4.6 Theoretical methods

There are several theoretical methods (Selim, 1971; Klute, 1952) and models available for predicting the moisture content in pavements but they are very complicated implicit numerical methods which require computer programs for solution. However, a short summary of the most important ones is relevant here.

Several investigators (Selim, 1971; Klute, 1952; Philip and De Vries, 1957; De Wet, 1965) have proposed mathematical formulae based on the thermodynamic principles for predicting moisture movement caused by non-isothermal and isothermal conditions. The literature indicates that the important liquid and vapour diffusivity parameters for defining the potential causing moisture movement in the formulae can be expressed quantitatively in terms of soil properties and soil suction.

Klute (1952) and Selim (1971) have developed models for predicting moisture movements in soils subjected to isothermal conditions. These models are based on finite difference solutions to the differential equations for one- and two-dimensional moisture movement.

Selim (1971) developed a two-dimensional transient and steady state water flow computer model for determining moisture content in soils. Numerical methods were used to develop the computer model. He solved the transient flow equation by using an implicit numerical method. The steady state flow equation was also solved by using a numerical approach. Selim was not able to compare his theoretical results with field data. However, he indicated that the theoretical flow patterns appear to agree with expected field patterns.

Elzeftawy (1974) compared measured water contents from both the laboratory and the field with moisture contents predicted by the model developed by Selim. It was evident from this work done by Elzeftawy that the model developed by Selim could be used to predict the isothermal field moisture content accurately with time.

Selim (1971) concluded that the implicit numerical method that he used for solving transient 2-dimensional wate. flow equations was convergent and stable and yielded results which appeared reasonable. The technique provides for the prediction of soil water content in a flow medium at any location and time if the soil properties are known. The technique is flexible and incorporate hysteresis, soil nonhomogeneity, changes in initial and boundary conditions and geometrical dimensions.

Richards (1965) has been fairly successful in using computer methods for predicting the time-space moisture conditions in pavement systems subjected to isothermal conditions. Richards used a two-dimensional model but he analysed only the isothermal condition. He also implemented a field study to check his model. He concluded that while his program provided a valuable tool for analysing subgrade moisture contents, there was inadequate field data for conclusions to be made about the accuracy of the model.

In 1970, Lytton and Kher developed a model to predict moisture movement in expansive clay subgrades. They analysed only the isothermal case, but they had both one- and two-dimensional programs, and used a field study to validate the model.

Figure 3.4 shows a typical comparison between field moisture content and predicted moisture for various depths below the surface. They concluded that excellent results could be achieved by use of their program provided the input data was of high quality.

(a) Geometrical values:

- (i) Number of pavement layers
- (ii) Depth of pavement layers
- (iii) Geometrical cross sectional design of roadway
- (iv) Depth of water table
- (v) Drainage system
- (vi) Free water conditions

(b) Temperature gradient

- (i) Temperature regime
- (ii) Thermal conductivity and heat capacity of soil and pavement materials
- (iii) Temperature fluctuation
- (iv) Frost line

(c) External moisture changes

- (i) Rainfall and infiltration
- (ii) Moisture movement from pavement edges
- (iii) Raising and lowering of water table

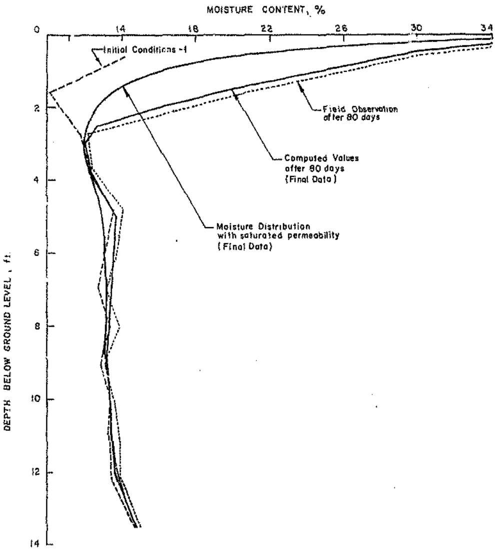

FIGURE 3.4

COMPARISON BETWEEN MEASURED AND COMPUTED MOISTURE CONTENTS AT A FIELD TEST SITE IN WYOMING. (LYTTON AND KHER, 1970)

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(d) Moisture movement variables

(i) The hydraulic conductivity (K)

(ii) Moisture diffusitivity (D_θ) - a measure of the force required to push water through soil and a function of water content and soil type

(iii) Temperature diffusivity (T_t) - measure of the force required to push water through soil and a function of temperature gradient.

A fairly accurate knowledge of all these variables is necessary to make an accurate prediction of the moisture content and this is obviously a tedious job. The aim of this dissertation is to find a very much simpler method of prediction even if some accuracy is forfeited.

3.4.7 <u>Comments on theoretical methods</u>

3.4.7.1 Limitations

These methods have the drawback that they are laborious to use. The mechanisms responsible for moisture fluctuations are very complex and seeing that the theoretical methods represent fundamental systems, they simulate these processes fairly accurately if accurate input data and boundary conditions can be provided. The drawback lies in the provision of these conditions. In many instances the input data are rough approximations which cannot be used in accurate methods.

Another limitation of the use of these methods is the condition of adequate waterproofing of the pavement. This is easy to fulfill only in cases of sandy soil subgrades whose coefficients of permeability K always exceed 10^{-4} cm/s and which are precisely those soils whose mechanical properties are the least sensitive to the moisture content. Silty soils ($K = 10^{-5} - 10^{-6}$ cm/s saturated) demand good quality dense pavements having a coefficient of permeability of about $10^{-6} - 10^{-7}$ cm/s, which means it must be free from previous cracks or joints and whose water proofing is not liable to be compromised during the useful life of the highway by damage due to wear, fatigue, rupture and excessive distortion. For clay soils, with K of the order of 10^{-8} cm/s, pavements with K's of 10^{-9} cm/s would be required.

Even if this condition is fulfilled, the isothermal conditions are generally not fulfilled, and water movements in the vapour phase can take place.

Even if the above conditions are fulfilled, the predictions of the theoretical method are not applicable to the edge zones of pavements. The width of the edge zone affected is approximately 1,2 - 2,0 metres. Water-proofing the shoulder can eliminate edge effects but it appears that processes used for this purpose have not yet proved their worth.

3.4.7.2 Accuracy obtainable

From the results of an experiment performed in Belgium it was possible to compare moisture contents measured in the subgrade of an open air model highway satisfying all conditions of application of the theoretical method, with moisture contents estimated by this method in relation to the depth of the groundwater level and the suction curves of the soil. The soil was a low plasticity silty soil (PI = 8) and it was relatively pervious ($K = 3 \times 10^{-5}$ at 10 ^{o}C for $S_r = 0,75$).

A stable pore water pressure profile was established four months after construction and the moisture contents remained constant at each level from that time over a period of 2 years of observations. It can therefore be concluded that equilibrium was achieved and maintained. The measured values proved on average to be 2 - 3 per cent lower than those predicted by the theoretical method. In other words, the soil drained slightly beyond the estimation obtained by the theoretical method.

It is true that the predictions were based on suction curves during drainage and determined in the laboratory from complete drainage. Initially the in-situ soil had a degree of saturation of about 0,75 only and it was certainly not being drained from complete saturation. A suction curve starting from the initial state should therefore have been used, which the measuring techniques available did not allow, but which would doubtless have shown a moisture content lower than the curve starting from complete saturation for the same suction.

Moreover, comparison of the measured and predicted moisture contents did not lead to very accurate conclusions because of variations of density which were observed throughout the soil mass, although this was quite uniform. This natural heterogeneity of soil masses nullified the gain in accuracy of predictions that, at first sight, could be expected from more advanced techniques of suction measurements.

From this experiment it was concluded that the equilibrium moisture content predicted by the theoretical method was close to the observed values, although somewhat higher.

3.4.7.3 Data required

Observations about the groundwater level are important. It is vital to ascertain the seasonal variations of this level and to spot the appearances of a false or perched water table. These observations, repeated along the entire length of the highway, are costly and take a long time.

In addition to the usual geotechnical measurements, the laboratory examination of the soil samples collected during the site survey should also include determination of the suction curves of the principal types of soil. The density of the test samples should correspond to the density provided for the compacted subgrade. The cost of determining these suction curves will be high and the time for their execution will inevitably be considerably longer than that required for ordinary geotechnical tests.

The project engineer wishing to use the rational method for evaluating moisture contents comes face to face with the problem of increased costs of the study and the longer time required for its execution. On the other hand, by way of compensation, he will be able to arrive at decisions such as modifications to the longitudinal profile and even to the location line liable to reduce appreciably the cost of the works and their duration of execution and thus make the longer preliminary work worthwhile.

3.4.7.4 *Value of the theortical method*

In spite of the limitations and difficulties of application which have been mentioned, the value of a theoretical method is undeniable. In the present state of knowledge this method is the only one to give reasonably accurate predictions, insofar as its applications are fulfilled.

Furthermore, the very fact that it is necessary to ascertain whether the method is applicable by examining the real characteristics of the site and of the highway from the viewpoint of hydrology (groundwater level), hydraulics (permeability of road structure and soil) and thermal (climatic) factors, will always provide useful information. Indeed, even if the conditions for application of the method appear not to be fulfilled, this examination will allow a prediction of the way in which the actual moisture profile might differ from the equilibrium profile. Thus it would provide the road designer with guidance in choosing measures to prevent dangerous moisture contents.

3.5 Conclusions

The following conclusions may be drawn from this chapter.

(a) Many conflicting observations about the general trends of moisture conditions in pavement subgrades have been made by various researchers.

(b) General agreement on the following trends emerged:

(i) The largest moisture variations occur within the first two years after construction, after which only relatively small seasonal variations occur.

(ii) Climate is only a major factor influencing the moisture regime when the water table is deep. Where the water table is shallow, it is the major factor influencing the moisture regime.

(iii) The material type and density also play a major part in influencing the ultimate moisture regime in as far as the moisture content increases with increasing plasticity and decreases with increasing density.

(iv) Moisture stability is reached beneath the greater part of the paved area (if the surfacing is relatively impervious) in all climatic regions.

(v) The maximum moisture conditions are reached towards the end of and shortly after the rainy season.

(c) Although there is an abundance of empirical prediction models available, it is clear that no generally acceptable model exists.

(d) The prediction accuracy of only a few models are known and it is clear that they generally give rather poor results.

(e) Rational prediction techniques have won practical acceptance in some countries, but they may not be applied to local conditions at this stage because they are generally based on characteristic moisture-suction curves, which are not available for local soils.

(f) Significant progress has been made in simulating the moisture regime mathematically, but it has found only limited practical application because of the uncertainty in the input values.

CHAPTER 4

INSTRUMENTATION AND COLLECTION OF DATA

CHAPTER 4

INSTRUMENTATION AND COLLECTION OF DATA

CONTENTS

CONTENTS CHAPTER 4 (Continued)

CHAPTER 4

INSTRUMENTATION AND COLLECTION OF DATA

4.1 Introduction

The aim of this chapter is two-fold. Firstly, a summary of available suction measuring techniques will be given, as well as a description of the specific apparatus used in the preliminary investigation into suction variations for this study. Where detailed descriptions of the apparatus are available elsewhere, the reader will be referred to the relevant literature and only a summarized version of the description and use will be given here. Where such descriptions are not available in the literature, a more detailed description is presented here. The instruments used to evaluate pavement response will also be summarized here. Secondly, the origin and nature of the southern African data used for comprehensive statistical analysis, is summarized.

Traditionally, in-situ suction measurements have been carried out using gypsum blocks, electrical resistance blocks and tensiometers, depending on the range of suction anticipated. In their statement on the symposium on moisture conditions in Australia in 1965, the review panel summarized the methods in general use at the time. They also gave an extensive list of references, indicating normal sensitivity, significant sources of error and ranges of applicability. At that stage they recognized the potential usability of psychrometers for engineering purposes. Later developments improved the reliability of this instrument and after an exploratory laboratory investigation, it was decided to use this instrument for the in-situ measurement of suction in this study. For completeness' sake, a brief summary of the main methods available to measure suction and moisture content is also given here.

All moisture content determinations for this investigation were done by the destructive method of taking samples and drying them in an oven, because soil samples had in any case to be taken at the same time.

There are various alternative methods and instruments available to measure in-situ moisture content, such as penetrometers, lysimetric methods, chemical, radar, radiometric and nuclear methods. Details of the alternative methods are reported by Cope and Trickett (1965), the OECD (1973), Ulaby (1974), Schmugge et al (1974), Eagleman (1976) and Visvalingam (1975).

4.1.1 The pF scale

The pressure required to remove water from an unsaturated soil is expressed in terms of the height in centimeters of a water column. The greater the cm height, the greater, of course, is the pressure measured. One complication, however, arises because some soils retain water with such tenacity that the suction necessary to remove it, if expressed directly in cm of water, gives awkwardly large numbers. For instance, certain increments of the colloidally adsorbed moisture require for their removal a force more than equivalent to the weight of a column of water 1 000 000 cm in height. To avoid using these cumbersome values, suction is expressed as the log of these values (Schofield, 1935). This log is called the pF and is analogous to the chemical use of pH for hydrogen ion concentration. Suction may also be expressed in atmospheres or bars - it is a matter of personal choice which system is used. Table 4.1 indicates the relation between pF and other parameters.

An observation of some engineering significance that can be made from this table is that the matric potential of the groundwater is approximately equal to the reciprocal of the mean diameter of the particles (both quantities expressed in cm).

4.2 Available methods of measuring soil water potential

4.2.1 Total potential

As explained elsewhere (Baver et al, 1972), total potential is

TABLE 4.1: THE RELATION BETWEEN pF AND OTHER PARAMETERS

pF	Height of water in cm	Diameter of particles in cm (for capillary action)	Negative pressure in ± atmospheres (bars)	Negative pressure in ± kPa	Negative pressure in ± PSI	Relative humidity (%)
0	1	1	,001	,1	,0147	-
1*	10	10^{-1}	,01	1	,147	-
2	100	10^{-2}	,1	10	1,47	99,99
3**	1 000	10^{-3}	1	100	14,7	99,92
4	10 000	10^{-4}	10	1 000	147	99,27
5	100 000	10^{-5}	100	10 000	1 470	93,00
6	1 000 000	10^{-6}	1 000	100 000	14 700	48,43
7	10 000 000	10^{-7}	10 000	1 000 000	147 000	0,07

Notes: * The soil liquid limit corresponds to a pF of 0,5 approximately (Croney and Coleman, 1954)

** The plastic limit corresponds to a pF of 2,8 to 3,3 approximately (Croney and Coleman, 1954) composed mainly of matric and osmotic components, if gravitational and external gas pressure potentials can be neglected. The majority of methods available to measure total water potential involve some way of measuring relative humidity.

Relative humidity is the ratio, usually expressed in per cent, of the pressure of water vapour in the gas to saturation pressure of water vapour at the temperature of the gas. In engineering, relative humidity is sometimes defined as the ratio of the weights, per unit volume, of the water vapour in the gas mixture and the saturated vapour at the temperature of the gas mixture. The two definitions are equivalent for all practical purposes.

Instruments used to determine relative humidity are called either psychrometers or hygrometers. Generally, the dry- and wet-bulb instrument is called a psychrometer, and direct indicators of relative humidity are called hygrometers.

4.2.1.1 Vapour pressure methods

These methods use the equilibrium humidity of an enclosed volume of air around a soil sample as a measure of the total suction in the soil.

Adsorption desorption techniques (vacuum desiccator, sorption balance)

Orchiston (1953) described the simple vapour adsorption/desorption techniques using H_2SO_4 solutions with soil in an evacuated desiccator. Various solutions have been experimented with and it has been established that the method has a range of 100 - 5 000 kPa of suction.

Several techniques have been developed which are based in the principle that the suction of a sample in equilibrium with a known humidity may be deduced from known relationships between humidity and suction. The test sample reaches equilibrium with the known humidity through moisture transfer in the vapour phase.

The relationship between humidity and total water potential is:

$$\phi = \frac{RT}{V} \ln \frac{e}{e_o}$$

This is the Kelvin equation.

where ϕ = total water potential in cm water

$\frac{e}{e_o}$ = relative humidity

R = universal gas constant

V = volume of mole of liquid water

T = absolute temperature (^{o}K)

This equation is based on the assumption that water vapour behaves as an ideal gas, which introduces only negligible errors.

Croney et al (1952) described in detail the apparatus and procedures whereby the suction-moisture relationship of a sample may be studied using either a sorption balance (pF 4,5 - pF 7) or a vacuum desiccator (pF 4,5 - pF 7). Both these methods are very sensitive to temperature gradients and care should be excercised to carry out measurements in a temperature-controlled laboratory.

4.2.1.2 Psychrometric methods

Spanner (1951) was the first to use Peltier cooling to measure the wet-bulb depression in a small chamber successfully, although techniques based on the measurements of the wet- and dry-bulb temperatures for determining relative humidities had been well established. Monteith and Owen (1958), Richards and Ogata (1958), Korven and Taylor (1959) and Klute and Richards (1962) have since used psychrometric methods successfully in soils. Rapid development of the instruments in the early 1970s produced a reliable and reasonably accurate instrument for field and laboratory work. They are now commercially available and have been used for in-situ suction measurements for this project, and will be discussed in more detail later (pF 2 - pF 5).

4.2.1.3 Freezing point depression method

This method is based on the fact that water absorbed by a porous material does not freeze at 0 ^{o}C in general, but at some lower temperature.

The freezing point depression is defined as the difference in temperature between 0 ^{o}C and the freezing point of the absorbed water. This depression is caused by both an increase in surface tension of the absorbed water and by dissolved salts. It is thus a measure or the total suction. Croney et al (1952) described the apparatus and procedure whereby this principle may be used to determine the suction in a sample in the laboratory. Limitations of the method is also discussed and it is clear that this method has not met with wide-spread application, either in the laboratory or in the field. It has, however, been used in the agricultural field and Cary and Fisher (1969) developed a freezing point meter to measure total plant water potential (pF 3 - pF 4).

4.2.1.4 Electrical methods (gypsum blocks, nylon blocks, resistivity probes, etc.)

These methods are based on the principle that the electrical resistivity of a soil varies with changing moisture content. A dry soil has very low electrical conductivity and hence a high resistivity. Pure water has the same characteristics, but once in contact with soil, the water will invariably dissolve certain substances to become conductive. Naturally the purity of the water added to the soil will influence the resulting decrease in resistivity, and this is one of the many disadvantages of this method (Edlefson and Anderson, 1941). Since the probe is affected by the percentage soluble salts in the solution, it gives an indication of total suction. This poses obvious practical problems as far as calibration is concerned. The probe obviously has to be calibrated specifically for the regime in which it is to be used. Early workers also found that the contact resistance between the electrodes and the soil may be erratic (Briggs, 1899; Gardner, 1898).

Initially Whitney et al (1897), Gardner (1898, and Briggs (1899) simply installed electrodes into the ground and took readings via a Wheatstone bridge. Unacceptable errors occurred due to variations in contact between soil and probes and this led MacCorkle (1931) and Edlefson and Anderson (1941) to carry out an experiment with electrodes, but the errors could not be reduced. Refinements and modifications were made to

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It was then decided to embed the probes into an absorbent material which would be buried in the soil and left to reach equilibrium with the surrounding material. Any changes in suction in the soil would then be reflected by equal changes in the suction in the absorbent material. Commercialized absorbent materials include gypsum, nylon and fibre glass and recently developed probes incorporate thermocouples so that the temperature may be measured and readings be corrected for temperature. Bouyoucos and Mick (1939) used standardized blocks of plaster of paris in which were set electrodes, connected by insulated wires to an AC Wheatstone bridge. Such blocks will measure suctions within the pF range 2,6 - 4,2 but improvements such as the use of nylon and fibre glass in blocks (Bouyoucos, 1949; Colman and Hendrix, 1949) and the use of concentric and spot-welded stainless steel mesh electrodes (Croney et al, 1951; Postlethwaite and Trickett, 1956) have improved durability and sensitivity of blocks and extended the range from pF 2 to pF 7. The OECD (1973) found that these instruments generally are not very accurate and only indicate the general trend of moisture movements.

The electrical properties of a soil at various radio frequencies were investigated by Ratcliffe and White (1930) and this method was used to measure soil moisture by Balls (1932) and Fletcher (1940). Temperature and hysteresis effects, and the power loss in a wet medium, add complications. A microwave meter comprising an oscillator, receiver and a calibrated attenuator is commercially available for determining the moisture content of samples (Anon, 1964; Pande and Pande, 1962). The sample is placed between receiving and transmitting antennae and the absorption related to moisture content. The advantage of the latter method is the speed of reading, but the heterogeneous nature of soil presents a limitation to in-situ use of micro- or radio-wave methods.

4.2.1.5 Hygrometers

Electrical and mechanical hygrometers are available which consist of hygroscopic materials that either change in physical dimensions or in electrical properties with changing moisture content. Human hairs are commonly used as sensing elements in hygrometers and they indicate relative humidity directly over a wide range of temperature, but reliability rapidly decreases as the ambient temperature decreases below freezing. At these temperatures, they indicate relative humidity in terms of the vapour pressure of supercooled water, not that of ice. A major disadvantage is the lack of stability of the calibration under usual conditions of use. Another disadvantage is the lag of hair hygrometers, particularly at low temperatures. Suddenly subjected, at about 25 ^{o}C, to a change in relative humidity, hygrometers take of the order of 5 minutes to indicate 90 per cent of the change in relative humidity; this time interval increases with decrease in temperature of the hairs.

Electrical hygrometers usually depend upon the change in electric resistance of a hygroscopic material with change in humidity. In one design, largely developed by Dunmore (1938), two parallel wires of a noble metal are wound round a polystyerene cylinder or strip. A hygroscopic coating is placed between the wires. At constant temperature, the logarithm of the electrical resistance between the parallel wires varies approximately linearly with the logarithm of the relative humidity and is measured by a suitable Wheatstone bridge, preferably using alternating current, or other suitable electric circuits. The electric resistance at constant relative humidity is highly dependent upon temperature, especially at temperatures below 0 ^{o}C.

4.2.2 Matric potential (suction)

4.2.2.1 Tensiometer, suction plate and centrifuge

A tensiometer consists of a waterfilled porous cup buried in the soil and connected to a pressure measuring advice. Water in the porous cup reaches equilibrium with the soil water and the pressure may be recorded continuously. Tensiometers are not intended primarily as a means of studying the suction-moisture content relationship, although they have been employed for this purpose. Only the matric suction is measured, as the cup is permeable to the solutes. Early designs are described by Richards (1949). Most are made by glueing a ceramic cup to a length of rigid plastic or copper tubing. If plastic tubing is used, it should be impermeable to both water and air. A gauge is inserted at the other end of the tube. Since diurnal variations in temperature affect the readings of tensiometers, especially those with parts exposed, care should be taken to read each instrument at the same time each day. Installation completely below the ground or a plastic construction using double walled tubes, will also reduce the influence of temperature. More elaborate and more sensitive tensiometers for laboratory measuremets have been described by Miller (1951) and Leonard and Low (1962). The chief disadvantage of the tensiometer is that it measures only relatively low suctions. It is normally only effective from pF 0 - ± pF 3.

The suction plate apparatus is based on the same principle as the tensiometer. The sample is placed on a moist porous plate where suction can be controlled. This is usually done by applying a suction by raising the sample above a water level with which it is in contact. The sample will either lose or take up water from the porous plate, until equilibrium is established. The suction of the sample is then known and the characteristic moisture-suction relationship can easily be established. Details about the operation and composition of this equipment, as well as a description of the requirements for the porous plate and the influence of pressure and temperature on the results, are given by Croney et al (1952). The apparatus can be modified so that it may be used for in-situ measurements, but this is not common. It only measures the matric suction and is only applicable in the range pF 0 to ± pF 3.

The centrifuge is based on the same principle as the tensiometer and pressure membrane, except that it may be used for much higher suctions because the test is carried out in a centrifugal field. If the distance between the centre of the sample and the water table is h, the suction exerted on the moisture in the sample is equal to a negative head of water h, in the earth's field, but in a field of ng the suction would be nh.

Thus in a field of 1 000 g, a sample 5 cm above the water level would give a suction of 5 000 cm of water, i.e. pF 3,7. Considerably higher gravitational fields than this may be achieved by spinning a sample at high angular velocities. Details about the apparatus and test procedure is given by Croney et al (1952). It has successfully been used in the range pF 3 - pF 4,5.

4.2.2.2 Pressure membrane method

The pressure membrane apparatus was developed by Richards (1941, 1947) to measure the uptake and release of moisture by samples of soil over a wide range of suction values. Far higher suctions may be measured than with a tensiometer and only the matric suction is measured.

The apparatus comprises a metal pressure chamber, the floor of which consists of a cellophane membrane supported by a porous metal filter so as to withstand high pressures from inside. This membrane will pass water and also dissolved salts (Reitemeyer and Richards, 1944), but will hold back compressed air when wet. Some leakage of air does occur (Collis-George, 1952) but this does not affect routine use of the method (Quirk, 1954). Croney et al (1952) described the use of this apparatus in detail so it will not be repeated here. Peck (1960) and Chahal and Yong (1964) described some errors that may occur when this apparatus is used. The range for which the apparatus is applicable depends on the porosity of the membrane. Materials such as cellophane and plastic sausage skins have been used successfully for suctions up to pF 4.

4.2.2.3 Thermal methods

Attempts have been made to exploit the thermal conductivity of the soil directly (De Vries, 1953; Van Duin and De Vries, 1954) and that of absorbent thermal blocks in close contact with the soil (Haise and Kelley, 1946) to derive soil moisture values in-situ. The thermal conductivity, unlike the electrical conductivity, of a soil is independant of the salt content. This presents a distinct advantage over resistance blocks, particularly for saline soils. Direct measurements depend upon very close contact between soil and probe, which is hard to achieve. However, De Vries (1953), whose method is based on the logarithmic rise in temperature with time of an infinite line source in a homogeneous medium, reports consistent results in sandy soil. Other techniques (Van Wijk, 1963; Talsma, 1964) have also been tried, with only limited success. Shaw and Baver (1939) used a glass tube with copper wire to measure moisture changes in-situ. Since the input to the element is constant, the resistance depends upon the thermal conductivity of the soil. In the method of Momin (1945) and Kubo (1953) a mercury thermometer having half the bulb wound with electrically heated wire, is set in the soil. The time required to obtain a constant temperature rise is said to be proportional to the soil moisture. These methods for obvious reasons have limited applicability.

4.2.2.4 Electrical suction piezometer

A fairly recent development in the measurement of in-situ suctions is the electrical suction piezometer. The cell was originally proposed by Escario (1967, 1969). As shown in Figure 4.1, the cell consists of a cylindrical porous disc through which equilibrium with the surrounding soil is established. On top of this is a thinner porous disc enveloped by membranes. This disc can be soaked with water from small water pipes leading to it. In close contact with this latter porous disc is the sensitive element which is a gypsum block with electrodes embedded in it. Above this element is a cavity filled with fibre glass and connected to a source of air pressure.

The cell is installed in the ground and a moisture equilibrium between the soil and the larger cylindrical disc is set up. When a reading is taken, water is applied at atmospheric pressure to the intermediate porous stone immediately below the sensitive element. At the same time air pressure is applied to the sensitive element through the porous fibre glass located on its upper part. On either side of the intermediate porous disc is a semi-permeable membrane, like cellophane, which permits slow passage of water but resists the applied pressure. If the applied pressure is equal to the water pressure in the sensitive element which is in equilibrium with the soil, then no water migration will take place. The electrodes in the sensitive element will indicate when water movement stops and the suction is then read off the applied pressure gauge.

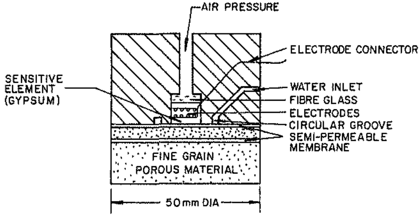


FIGURE 4.1

ELECTRICAL SUCTION PIEZOMETER

The OECD (1973) describes the d.tailed procedure for using this apparatus for different ranges of suction and also discusses the main problems involved. It can be used in the suction range pF 3 - pF 5.

Figure 4.2 shows a comparison between the different methods available to measure suction and also the range over which each instrument is operative.

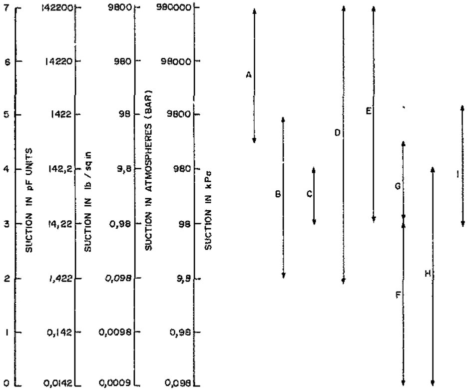
4.3 Instruments used for this investigation

4.3.1 Peltier thermocouple psychrometer

4.3.1.1 Theory

Because the operation of the Peltier psychrometer is restricted by the laws of thermoelectricity, a review of these laws will illustrate the operational limitations of the probe.

Firstly, the law of the homogeneous circuit states that an electrical current cannot be sustained in a circuit of a single metal by the application of heat alone. With respect to Peltier and Seebeck circuits, this means that the electromotive force (emf) developed at the junctions is independent of the spatial temperature distribution along their leadwires in the field. Secondly, the law of intermediate metals states that the sum of thermo-electromotive forces in a circuit composed of any number of dissimilar metals is zero if all the junctions are in thermal equilibrium. This is of particular significance regarding the Peltier psychrometer, since it is composed of 3 different metals (copper, constantan and chromel). This also allows the introduction of a meter into the circuit to measure the emf in the junctions of the meter with the circuit at the same temperature. Thirdly, the law of successive temperatures states that if a Seebeck thermocouple produces a net emf_1 when the junctions are at temperatures T_1 and T_2 and a net emf_2 when the junctions are at temperatures T_2 and T_3, then the net emf generated when the junctions are at T_1 and T_3 will be $emf_1 + emf_2$. An understanding of these laws and of the Peltier and Seebeck effects is essential to grasp the operation of the psychrometer.



A = VACUUM DESICCATOR AND SORPTION BALANCE
B = THERMOCOUPLE PSYCHROMETER
C = FREEZING POINT DEPRESSION
D = GYPSUM BLOCKS
E = ELECTRICAL RESISTANCE GAUGE
F = SUCTION PLATE AND TENSIOMETER
G = CENTRIFUGE
H = PRESSURE MEMBRANE
I = ELECTRICAL SUCTION PIEZOMETER

FIGURE 4.2

(Adapted from Croney et al 1952)

COMPARISON OF DIFFERENT SUCTION MEASURING TECHNIQUES

Psychrometers are used to infer water potential from measurements of relative humidity. The relationship between these at a given temperature is given by equation 4.1. When the relative humidity of the air space is in equilibrium with the vapour pressure of the pore water in the soil specimen, the suction may be evaluated from equation 4.1. The ambient temperature T substituted in equation 4.1 need not be known very accurately ($\pm$ 1,5 $^{\circ}$C) to infer the suction to the nearest 1 per cent (Rawlins and Dalton, 1971). The relative humidity, on the other hand, must be known very accurately, especially for humidity exceeding 95 per cent, because small errors in relative humidity amount to large errors in the natural logarithm of the humidity. Large errors in suction will thus result from equation 4.1.

The thermocouple psychrometer can accurately determine the difference between the dew point (wet-bulb) and ambient (dry-bulb) temperatures. The relative humidity is evaluated from this temperature difference and the ambient temperature. In this case one thermocouple junction is cooled by applying a small direct current in the proper direction through the junction, which causes cooling as a result of the Peltier effect.

The exchange of heat between a junction and its surroundings is called the Peltier effect. For small currents the cooling is proportional to the current, as shown in Equation 4.2

$$\frac{dH}{dt} = -\Pi_{ab} I + RI^2 \quad \ldots\ldots\ldots\ldots \quad (4.2)$$

where $\frac{dH}{dt}$ = rate of change of heat

Π_{ab} = Peltier coefficient

I = applied current

R = resistance.

It is clear that as the current increases the junction will heat up, not cool down, because of the influence of the square of the current. There is thus an optimum cooling current that should be applied. This is achieved by the electronics. Water starts to condense on the thermocouple junction (wet-bulb) when the temperature reaches the dew point, inhibiting further cooling below the dew point temperature. After the cooling current has been terminated, the water on the wet junction begins to evaporate and a difference in temperature is maintained between the wet and dry junctions until the water has evaporated.

The drier the atmosphere, the more rapid will the evaporation rate be and hence the greater will the temperature differential between the wet and dry sensors be. This temperature difference between the junctions causes a small voltage to be generated according to the Seebeck effect. The Seebeck effect states that if two dissimilar metals are joined together in a closed circuit, then an electrical current will flow in the circuit if the two junctions are at different temperatures.

Chromel-constantan thermocouples were used in this investigation because they yield a relatively large voltage output for small temperature changes and they resist corrosion. The junction may also be heated sufficiently to burn off excess moisture before the next measurement is taken. The output voltage may now be related to total suction from calibration curves.

Several questions have arisen covering the validity of relative humidity measurements using these thermocouple psychrometers. They are used in static air, for instance, whereas psychrometer wet-bulbs are normally aspirated to achieve accurate readings. It has also been found that the wet-bulb depression of the psychrometer is not a unique function of the relative humidity of the chamber in which it is placed. Sensitivity is found to increase with increasing temperature and decrease with increasing pressure. Rawlins (1966) derived a theoretical equation relating the output voltage to psychrometer dimensions and to chamber temperature and pressure. This equation agrees well with experimental results and it is clear that it takes all the major factors affecting psychrometer performance into account. The previously raised objections now become insignificant, because it is possible to infer the relative humidity within an equilibrium chamber from the output of the psychrometer calibrated at another temperature and pressure. The interested reader is referred to Tanner (1972), Rawlins (1972), Rose (1966), Harrison (1965) and Tanner (1963) for more complete and rigorous discussions of psychrometric theory.

Although the majority of objections regarding the psychrometric method have been dispelled, all suction measurements taken for this investigation had also been done by the dew point method. The chief advantage of the dew point method is that no net water exchange occurs at the wet junction, allowing measurements to be taken without disturbing the vapour equilibrium.

It is thus relatively insensitive to all factors affecting vapour exchange, like size and shape of wet surface at the junction (Neumann and Thurtell, 1972). The dew point method is based on the phenomenon that the dew point temperature is depressed as the water potential is reduced below saturation (becomes more negative). Hypothetically a wet junction will always tend to the dew point temperature, because, if it is below it, extra water will condense and raise the temperature, and, similarly, if it is above, water will evaporate, bringing the temperature down. External heat transfer mechanisms prevent this from happening in nature. During actual measurements, the wet junction temperature will always be below the ambient temperature and therefore heat will flow from the surroundings to the junction. It is possible to apply a Peltier cooling current, adjusted electrically, to exactly balance the heat inflow for a net zero heat transfer. This is done by applying a pulsed cooling current, which is controlled by the output of the microvoltmeter so that it balances the heat inflow. The junction temperature is thus initially lowered to below the dew point in order that a film of water be condensed onto it. When it is switched to the dew point position, the cooling current is immediately reduced from its maximum value to a value dictated by the microvolt output, such that heat is removed from the junction at the same rate it flows in from the surroundings. Water starts to condense and, through the heat of condensation, raises the junction temperature. This reduces the microvolt output, which in turn adjusts the cooling current. This automatic process continues until the junction temperature reaches the dew point and condensation ceases. The system will then automatically reach the dew point temperature. However, if a change in ambient temperature occurs, the meter output will be affected proportionately, since the initial ambient temperature is used as a reference temperature from which the dew point depression is measured. For more detailed information the reader is referred to Campbell et al (1973) and Neuman and Thurtel (1972).

4.3.1.2 Construction

The construction of thermocouples and sample holders for Peltier psychrometers has been described in several publications (Spanner, 1951; Monteith and Owen, 1958; Campbell et al, 1966; Merril et al, 1968;

Brown, 1970) and only a summarized description of the construction will be given here.

The t...rmocouples are constructed from chromel and constantan wires for reasons discussed before. Because it is desirable to cause as little thermal and vapour disturbance in the chamber as possible, the junctions should be as small as possible. The output for a large diameter thermocouple for a given suction is also less, which implies that the smaller diameter thermocouples are more sensitive and accurate (Johnson, 1974). Thermocouple wire with a diameter of 0,025 mm is commercially available.

Figure 4.3 shows the essential parts of the psychrometer probe. It consists of a small (48 mm di.meter, 65 mm long) teflon body through which two small (0,25 mm) holes are made with a dissecting needle. The copper lead wires are attached to the thermocouple through these holes. The chromel and constantan wires are soldered to the copper leads, which are then pulled back into the teflon body so that only the thin thermocouple wires protrude. These wires are then twisted together and the excess cut off. The thermocouple is now ready for welding. Welding is preferred because soldered junctions age more quickly and do not have a linear response throughout the range of vapour pressures of interest (Campbell <u>et al</u>, 1968).

From the figure it is clear that the psychrometer has two reference junctions instead of one (copper-constantan and chromel-copper) and only one sensing junction (chromel-constantan). When these three junctions are in thermal equilibrium, the psychrometer has a net output of zero volts. The sum of the emf's generated per ^{o}C at the two reference junctions ($\pm$ 40 µV $^{o}C^{-1}$ at the copper-constantan plus $\pm$ 20 µV $^{o}C^{-1}$ at the chromel-copper) is exactly equal but opposite in sign, to that generated by the sensing junction (about 60 µV $^{o}C^{-1}$ at the chromel-constantan junction). A tight-fitting cup is fitted to the teflon body and the unit sealed with shrink sleeving. A copper-constantan temperature thermocouple is attached close to the probe with shrink sleeving. The unit is now sealed and ready for installation.

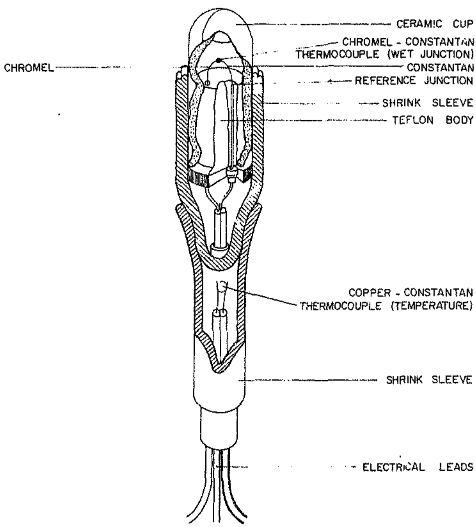


FIGURE 4.3

PT51 - 10 PELTIER THERMOCOUPLE SOIL PSYCHROMETER

940 - 4 - 2738 L.B.

4.3.1.3 Operation

The probe is installed into the pavement by making the smallest practical hole through the pavement, usually by drilling, to the required depth. The probe is bedded in the material retrieved from the excavation, if it is fine-grained, or in a small amount of clean-washed sand if the in-situ material is gravelly. The probe must be installed horizontally, especially if it is close to the surface, because there exists a vertical temperature gradient and the probe is very temperature-sensitive. The hole is backfilled with the material retrieved from excavation and care must be taken to ensure that good compaction is achieved to prevent the formation of a channel for easy water ingress. It is sometimes necessary to seal the hole between successive pavement layers.

If the ceramic cup had been wetted with distilled water prior to installation, equilibrium will have been achieved after 24 hours under most circumstances. Otherwise a few days may be necessary.

The probes may be monitored by any microvoltmeter in the psychrometric mode, but when the dew point mode is used, then sensing and switching needs to be incorporated. Microvoltmeters which contain these features are commercially available. For details regarding taking of measurements and connecting-up procedures, see Haupt (1978a). The probes have proved themselves to be reasonably reliable and accurate in this study for suction ranges pF $\pm$ 2 to pF $\pm$ 5. The probes in Namibia remained serviceable for periods well in excess of 12 months while some probes near Stormwater in the Cape became unserviceable after only 2 months.

4.3.2 Resistivity probes

The principle governing resistivity probes is that an electrical voltage is applied to the ground, and the resistance to flow of an electric current is measured. The probe thus has to comprise both a generator and a detector. The resistivity of soil depends on the quantity of water retained in the pores and electricity is conducted by the process of electrolyses. Figure 4.4 shows the construction of the NITRR resistivity probe.

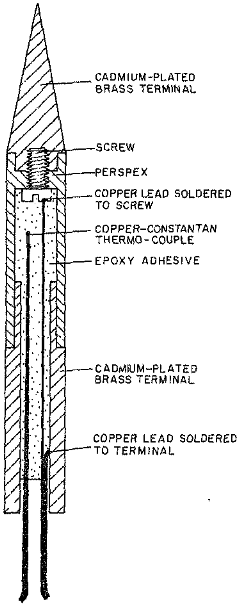


FIGURE 4.4

NITRR RESISTIVITY PROBE

140-4-2750 B.D.

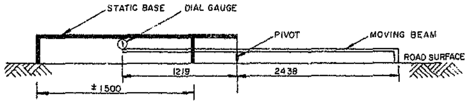
The main disadvantage of this probe is that the resistance is influenced by the amount of dissolved salts in the solution and that a calibration curve for each probe in each material is necessary. This rendered its application in this study very limited. It was only used to detect general qualitative moisture trends.

4.3.3 Benkelman beam

The Benkelman beam is used to measure the road surface deflection. It consists basically of a static base which supports a moveable beam from its third point which, when [illegible] end is placed between the dual wheels of a loaded truck, can [illegible] tical deflections in the road surface at that point to a dial gauge [illegible] d on the static frame. Figure 4.5 shows a schematic lay-out of the apparatus. This instrument was successfully applied to measure the pavement response. It was, however, found that because it only measures the total surface deflection, very little could be said about the individual layers' contribution to the total deflection. It was felt that for the more detailed study of diurnal suction variations in specific pavement layers, and the contribution of specific layers' performance to total pavement response, multidepth deflectometers should be used. More work is at present being planned in this regard and these latter instruments will in fact be used to evaluate the pavement response in more detail, instead of the Benkelman beam and the radius of curvature meter. Preliminary work has already been done and foolproof installation procedures developed.

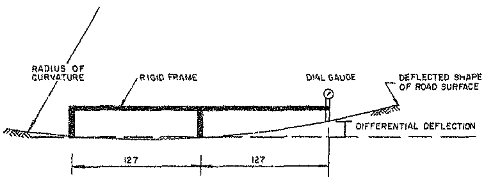
4.3.4 Radius of curvature meter

The NITRR curvature meter consists of a short bar standing on fixed feet. The spindle of a dial gauge passes through a hole at one end of the bar so that the dial can deflect freely with any change in the road surface. The point of the spindle and the point of the outer foot are 254 mm apart with the other foot midway between them. The gauge indicates the differential deflection between these three equally spaced points on the road surface (Figure 4.6).



NOTE: ALL DIMENSIONS IN MILLIMETRES

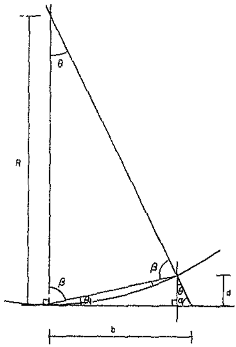
FIGURE 4.5 *PRINCIPLE OF BENKELMAN BEAM*



NOTE: VERTICAL SCALE HIGHLY DISTORTED
ALL DIMENSIONS IN MILLIMETRES

FIGURE 4.6 *PRINCIPLE OF CURVATURE METER*

From this measurement of relative deflection, the radius of curvature of the deflection bowl may be calculated as follows:



$$\beta = \theta_1 + \alpha$$
$$2\beta + \theta = 180$$
$$\therefore \beta = 90 - \frac{\theta}{2}$$
$$90 - \frac{\theta}{2} = \theta_1 + \alpha$$
$$90 - \frac{\theta}{2} = \theta_1 + 90 - \theta$$
$$\theta_1 = \frac{\theta}{2}$$

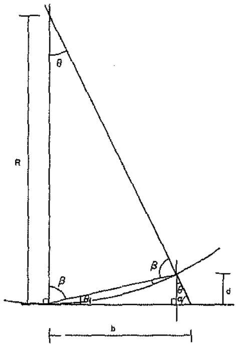
Using the small angle approximation $\theta = \tan,\theta = \sin\theta$

$$\theta = \tan\theta = \frac{b}{R}$$

$$\theta_1 = \tan\theta_1 = \frac{d}{b} \quad (R >> 76)$$

$$R = \frac{b^2}{2d}$$

From this measurement of relative deflection, the radius of curvature of the deflection bowl may be calculated as follows:



$$\beta = \theta_1 + \alpha$$

$$2\beta + \theta = 180$$

$$\therefore \beta = 90 - \frac{\theta}{2}$$

$$90 - \frac{\theta}{2} = \theta_1 + \alpha$$

$$90 - \frac{\theta}{2} = \theta_1 + 90 - \theta$$

$$\theta_1 = \frac{\theta}{2}$$

Using the small angle approximation $\theta = \tan \theta = \sin \theta$

$$\theta = \tan \theta = \frac{b}{R}$$

$$\theta_1 = \tan \theta_1 = \frac{d}{b} \quad (R >> 76)$$

$$R = \frac{b^2}{2d}$$

The general formula is $R = \frac{b^2}{Fd}$ where F is a factor to account for the specific shape of the deflection bowl. NITRR assumes a sinusoidal shape instead of a circular shape, the curvature meter measures 2d and the dial gauge on the curvature meter is usually calibrated in 0,01 mm and so the formula used to yield the curvature in meters is $R = \frac{1\ 400}{d}$.

4.4 Origin and nature of data used in this study

One of the aims of this study was to collect all available local moisture data in order to evaluate the existing moisture prediction models and establish new models if necessary. All the road authorities in southern Africa were therefore requested in 1977 to supply all the then available data pertaining to this problem. The data supplied by the different authorities is summarized below.

4.4.1 Transvaal data

The Transvaal Roads Department had carried out two moisture investigations of existing surfaced roads before 1977. The first was conducted between 1965 and 1967, when moisture determinations were periodically done on some 30 sites throughout the Transvaal. These results were reported on by Mr G.P. Marais in the discussion of the 4th Regional Soil Mechanics Conference for Africa in 1967.

The second investigation was carried out in 1973 (Burrow, 1975), not for the purpose of studying moisture conditions, but to meet the following aims:

(a) to provide criteria for the pavement design of future highways in the Transvaal, based on the performance of the existing roads; and

(b) to establish the relative performances of the different types of base materials.

Roads were sampled in all districts throughout Transvaal in order to cover as many varieties of climatic, geological and traffic conditions as possible. A total of 1 102 holes were drilled in the pavements and all the layer thicknesses measured. Only the subgrade was sampled and a complete laboratory analysis carried out on the soil. The following routine soil tests were done according to the standard test methods of the Department of Transport (1971), modified as indicated:

(a) Sieve analysis (wet): test no. A1(a)

(b) Soil constants: I - grading modulus

$$\left(3 - \frac{(\% - 2{,}00\ \text{mm}) + (\% - 0{,}425\ \text{mm}) + (\% - 0{,}075\ \text{mm})}{100}\right)$$

II - liquid limit - test no. A.2
III - plasticity index - test no. A.3
IV - bar linear shrinkage - test no. A.4
V - sand equivalent - test no. B.19
VI - classification

(c) Maximum dry density: test no. A.7 but with Proctor compaction

(d) Optimum moisture content: test no. A.7 but with Proctor compaction

(e) Maximum [illegible]ifornia Bearing Ratio: test no. A.8 but on same moulds as (c) a[illegible] [illegible].

To include the possible influence of climate on the moisture regime, the following climatic indices were evaluated for each borehole site:

(a) Mean annual rainfall These values were obtained from the mean annual rainfall map which is based mainly on data from the 30 years period 1921-1950. The values were recorded in decimeters to an accuracy of MAR $\pm$ 0,25 decimeters.

(b) Weinert's N-value (Weinert, 1974) Although the climatic N-value was not developed to describe a climate, it provides a useful numerical expression relating important climatic factors. It is expressed as twelve times the computed free water evaporation during the warmest month divided by the total annula precipitation. Maps of climatic N-values for southern Africa are readily available and the one used in this study was "Contour Map of Climatic N-values for southern Africa", available from NITRR (RM 1 - 59A). Because a change in N-value from 1 to 2 represents a much larger change in climate than a change from 5 to 6, only lines 1, 2, 3, 4, 5, 7, and 10 were drawn. The accuracy of inte[illegible]lation was N-value $\pm$ 0,1 over the whole range.

(c) Thornthwaite's I-value Thornthwaite (1948) developed an index in an attempt to rationally classify climate. The background to the development of this index is well documented in the literature and is outside the scope of this dissertation. The index used here is the moisture index, which combines the indices of humidity and aridity and is expressed as

$$I = \frac{100s - 60d}{n}$$

where I = Thornthwaite's moisture index

s = the water surplus in units of length

d = the water deficiency in units of length

n = the water need in units of length

The Transvaal comprises climates from I = -40 to I = 100, which can be classified as semi-arid to humid regions. The I-value for each site was interpolated with an accuracy of I-value $\pm$ 2,5.

The physical properties of each site were also described and noted down.

A typical field data sheet is included in Appendix A, which also shows the format of the data base used in this analysis. The following features were recorded:

(a) Exact position of the site

(b) Time of sampling

(c) Elevation of road surface relative to the surroundings. Seventy per cent of all holes were drilled in flat terrain.

(d) Condition of surface drainage. In fifty per cent of the cases the surface drainage facilities were rated as fair, with twenty per cent rated as good and thirty per cent as bad.

(e) Slope erosion

(f) Road condition in vicinity of borehole. In seventy per cent of the cases the pavement condition was rated as "not failed".

(g) Surface condition in vicinity of borehole. In sixty per cent of the cases the surface was not cracked

(h) Description of each layer. In ninety per cent of the cases the thickness of bitumen surfacing was less than 50 mm (mean = 35 mm). Also in ninety per cent of the cases the total pavement thickness was 600 mm or thinner (mean = 410 mm).

(i) Moisture content of the subgrade.

(j) General remarks.

The number of valid cases and the minimum, maximum and mean values of the other parameters used in the analysis are summarized in Table 4.2.

From Table 4.2 it is clear that a large number of samples had been tested, covering a wide range of material and climatic conditions. It should also be pointed out here that the moisture contents measured were assumed to be equilibrium moisture contents, because all the pavements were older than 5 years, and because substantial evidence exists (Black et al, 1959; Scala, 1962; Russam, 1962; O'Reilly et al, 1968; Aitchison and Richards, 1965; Richards 1967) to indicate that equilibrium conditions do occur under pavements in similar climatic regions. A study of daily and seasonal moisture content variations to check this assumption for local conditions is still in progress. Even if this is not the case, the models developed here may be used to estimate probable maximum and minimum values as explained later in Chapter 9.

4.4.2 Rhodesian data

The Ministry of Roads and Road Traffic in Rhodesia had also conducted moisture investigations. An investigation into moisture and density conditions under pavements in Rhodesia was started in the early 1960's. Only one report was available on the findings of this investigation and it includes very valuable information similar to that of the Transvaal Roads Department. Unfortunately only five sites were sampled, which was probably sufficient for the specific needs at the time, but they would have little influence when analysed together with Transvaal's 1 102 sites.

Van Der Merwe (1967) also reported on the equilibrium moisture content of subgrades in a paper to the 4th Regional Soil Mechanics Conference for Africa. The effects of various factors on the equilibrium moisture content were discussed, but no information was given on the physical state or routine tests of the site. The regression equation he developed on that data performed disappointingly with local data, as discussed in the next chapter.

Investigations (Van der Merwe, 1967; Russel and Patel, 1976; Mitchell and Venter, 1974; Mitchell and Northrop, 1973) were also carried out at the Salisbury and Victoria Falls airports. The aims were to determine why the base wetted up, to monitor the water and temperature regime beneath asphaltic seals and to check for hydrogenesis. This data could also not be used, because it was not as complete as the Transvaal data, which formed the data base.

4.4.3 South West Africa, Orange Free State, Natal and Cape data

No data apart from as-built construction records and soil survey sheets were available from these authorities.

4.4.4 NITRR data

After various useful prediction techniques had been developed from the Transvaal data, it was felt that they cound not summarily be applied to any climatic region in southern Africa. It was then decioed to carry out two small-scale moisture studies in two areas which represent climatic extremes, viz. the Keetmanshoop area in Namibia and the Storms River area in the Cape. From the analyses of the Transvaal data it could be established that, for the proposed sites to be representative of important predicting parameters, they would have to include:

(a) ten different subgrade soil types representing a range of optimum moisture content values from ± 4 to ± 50 per cent;

(b) three different road conditions for each soil type above, i.e. cut, fill and flat areas.

This investigation was carried out early in 1978 and 31 different sites were sampled near Storms River while 29 different sites were sampled in Namibia. The information obtained from each site was identical in format to that of the Transvaal investigation. The physical conditions at each site were fully described and samples taken to be tested in the laboratory as explained in Section 4.4.1. Some results of this study are summarized in table 4.2.

TABLE 4.2: SUMMARY OF DESCRIPTIVE STATISTICS

	Number of valid cases			Mean			Minimum			Maximum		
	SWA	Tvl	Cape	SWA	Tvl	Cape	SWA	Tvl	Cape	SWA	Tvl	Cape
Equilibrium moisture content	28	1072	30	5,75	10,50	19,74	2,9	1,2	4,4	11,6	38,9	104,0
Grading modulus	28	1091	30	1,480	1,239	0,592	0,70	0,10	0,04	2,69	2,57	1,47
Liquid limit	25	996	19	25,64	29,02	25,62	18	13	12	51	88	51
Plastic Limit	25	993	19	14,08	16,15	14,68	10	8	7	20	41	25
Plasticity index	28	993	19	10,39	12,91	7,27	0	1	0	31	57	28
Bar linear shrinkage	28	1077	30	4,79	5,51	2,88	0	0,3	0	10,0	21,7	12,0
Sand equivalent	28	1055	30	20,25	14,38	13,63	9	2	0	55	46	70
Proctor maximum dry density	28	1086	20	2021,0	1887,9	1821,7	1784	1311	1517	2234	2396	2035
Proctor optimum moisture content	28	1086	20	10,46	12,88	14,0	6,9	6,1	8,4	18,5	31,2	24,1
Maximum CBR	28	1084	20	29,20	20,33	30,53	4,0	0,9	3,0	62	120	72
% - 0,425 mm	28	1079	30	51,93	63,09	89,47	10	11	58	94	99	100
% - 0,075 mm	28	1079	30	28,57	35,85	55,93	7	4	13	66	91	96
MAR	28	1086	30	138,90	690,20	993,20	110	400	700	180	1500	1230
N	28	1086	30	44,40	3,30	1,00	32,0	1,0	1,0	50,0	7,6	1,0
I	28	1086	30	-40,00	- 4,79	10,00	-40	-45	10	-40	100	10

4.5 Conclusions

The following conclusions may be drawn from this chapter.

(a) The recent developments in electrical psychrometer probes have made these instruments reliable enough for the monitoring of suction on a routine basis in the field.

(b) If suctions below pF 2 are expected, then tensiometers should be used for field and laboratory measurements, as psychrometers are inaccurate in this range.

(c) The useful life of a psychrometer depends primarily on the moisture régime in which it is used. In a relatively dry environment the probes easily last longer than 12 months. In a continuously wet climate they may only last 2 or 3 months.

(d) It is recommended that for more detailed investigation into the diurnal suction variation in the respective pavement layers and the associated variation in pavement response, multi-depth deflectometers should be used.

(e) Gypsum blocks, nylon probes and resistivity probes were found not to be suitable for accurate routine field suction measurements.

(f) A large number of samples, covering a wide range of material and climatic conditions, had been used in this analysis, rendering the conclusions drawn reliable and applicable to a wide variety of conditions.

CHAPTER 5

EVALUATION OF EXISTING MODELS AND GENERAL LOCAL TRENDS

CHAPTER 5

EVALUATION OF EXISTING MODELS AND GENERAL LOCAL TRENDS

CONTENTS

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CONTENTS CHAPTER 5 (Continued)

CHAPTER 5

EVALUATION OF EXISTING MODELS AND GENERAL LOCAL TRENDS

5.1 Introduction

Although it was concluded at the end of Chapter 3 that no single empirical model had won widespread acceptance and that the prediction accuracies of very few models were known, it was nevertheless decided to test these models for local conditions before developing other models. The first part of this chapter describes this testing procedure, which was carried out as follows:

(a) Firstly, each model was applied to local data to determine how applicable it was.

(b) Secondly, a regression analysis was carried out on the local data using the same soil parameters as those used in the original model in an attempt to modify it to suit local conditions best. The original model is always denoted by PEMC for "predicted equilibrium moisture content" (plotted as solid lines in figures) and the modified model by EMC for "equilibrium moisture content" (plotted as broken lines in figures).

The abbreviations used in equations and figures are explained in Appendix A.

The second part of this chapter summarises the subgrade moisture trends in southern Africa as they emerged from a thorough statistical analysis of all available data to date. This investigation was carried out because it was concluded in Chapter 3 that many conflicting observations had in the past been made by various researchers. It should be remembered that these are general trends and should not blindly be applied to any situation.

5.2 Evaluation of existing models

The results of the evaluation exercise are reported in this chapter, and more detailed information is contained in Appendix B.

5.2.1 Thornthwaite moisture index method (Thornthwaite, 1948; Russam, 1962)

This method is really a combination of the empirical and rational methods and because the soil moisture characteristic curves need to be known, this method could not easily be tested for local conditions without a fair amount of laboratory work to establish these curves.

5.2.2 Swanberg and Hansen model (Swanberg and Hansen, 1946; OECD, 1973)

The model proposed by these researchers was as follows:

$$PEMC = 1{,}16(PL) - 7{,}4 \quad \ldots\ldots \quad (5.1)$$

where PEMC = predicted equilibrium moisture content as proposed by Swanberg and Hansen.

When tested for local conditions, this model yielded a standard error of estimate of 4,804 which makes it poorly applicable. Figure 5.1 indicates the wide scatter of the predicted values around the actual values. The best model, using PL for local conditions, is:

$$EMC = 0{,}7762(PL) - 1{,}383 \quad \ldots\ldots \quad (5.2)$$

Figure 5.2 shows the improved accuracy of equation (5.2) over equation (5.1).

Because the majority of samples used for this original investigation had plastic limits between 15 and 30, this model was also tested for local conditions for these limits and a standard error of estimate of 5,465 was found. Although the statistics given in Figure 5.3 are not directly comparable with the previous statistics, because of a different range, it can be deduced that no significant improvement has resulted.

5.2.3 Haliburton model (1971)

Haliburton found that the equilibrium moisture content usually reaches a value between 1,1 and 1,3 times the plastic limit. Only five per cent of local values ever exceed this upper limit, as can be seen in Figure 5.4, which means that this model may be used as an upper limit.

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FIGURE 5.1

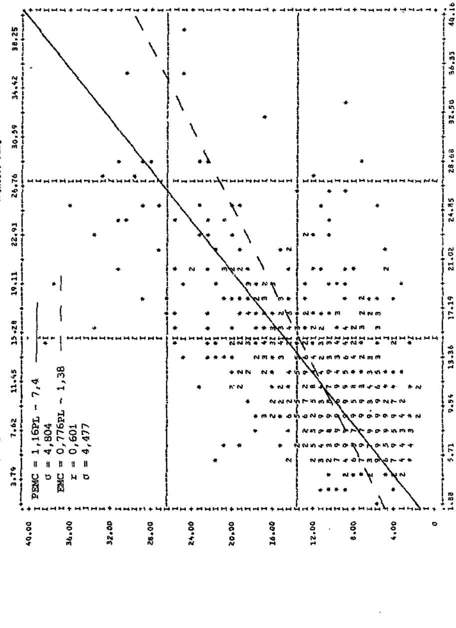

SCATTERGRAM OF (DOWN) EMC (ACROSS) PEMC
PEMC = 1,16PL - 7,4
σ = 4,804
EMC = 0,776PL - 1,38
r = 0,601
σ = 4,477
3.79 7.62 11.45 15.28 19.11 22.93 26.76 30.59 34.42 38.25
1.88 5.71 9.54 13.36 17.19 21.02 24.85 28.68 32.50 36.33 40.16
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0

FIGURE 5.1

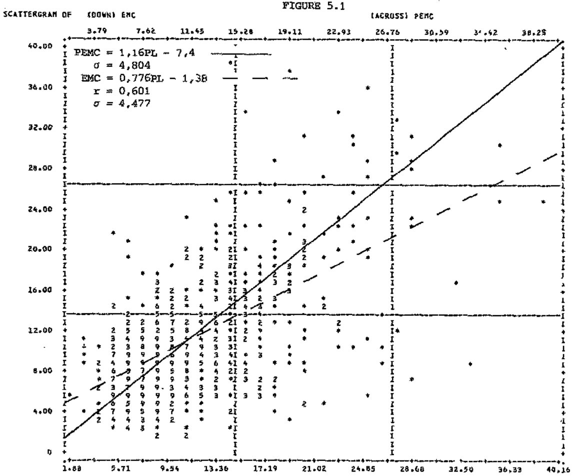

SCATTERGRAM OF (DOWN) EMC
(ACROSS) PEMC
3.79
7.62
11.45
15.28
19.11
22.93
26.76
30.59
3*.42
38.25
40.00
36.00
32.00
28.00
24.00
20.00
16.00
12.00
8.00
4.00
0
1.88
5.71
9.54
13.36
17.19
21.02
24.85
28.68
32.50
36.33
40.16
PEMC = 1,16PL - 7,4
σ = 4,804
EMC = 0,776PL - 1,38
r = 0,601
σ = 4,477

FIGURE 5.2

SCATTERGRAM OF (DOWN) EMC (ACROSS) PL

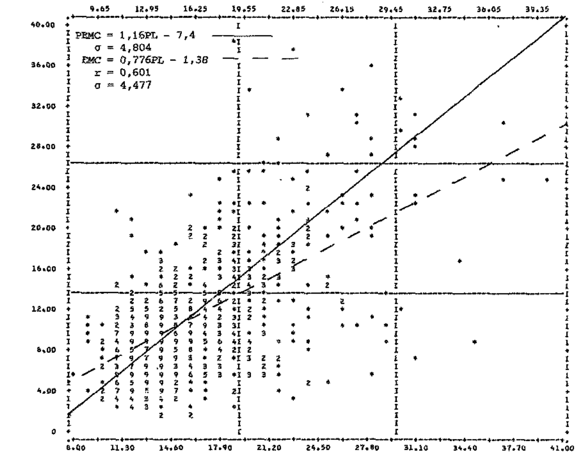


FIGURE 5.3

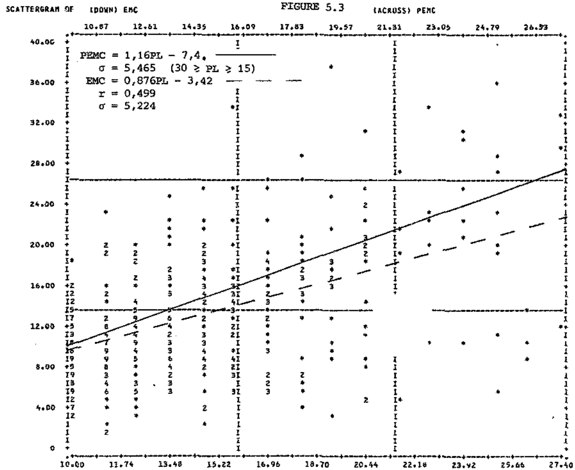
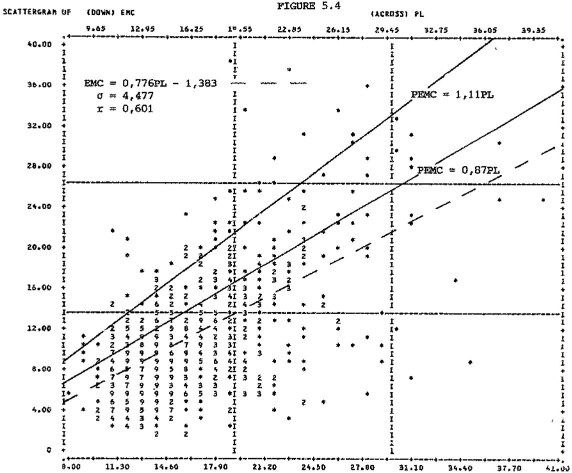

SCATTERGRAM OF (DOWN) EMC (ACROSS) PEMC
10.87 12.61 14.35 16.09 17.83 19.57 21.31 23.05 24.79 26.53
PEMC = 1,16PL - 7,4,
σ = 5,465 (30 ≥ PL ≥ 15)
EMC = 0,876PL - 3,42
r = 0,499
σ = 5,224
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0
10.00 11.74 13.48 15.22 16.96 18.70 20.44 22.18 23.92 25.66 27.40

FIGURE 5.4


SCATTERGRAM OF (DOWN) EMC
(ACROSS) PL
EMC = 0,776PL - 1,383
σ = 4,477
r = 0,601
PEMC = 1,11PL
PEMC = 0,87PL
9.65
12.95
16.25
19.55
22.85
26.15
29.45
32.75
36.05
39.35
40.00
36.00
32.00
28.00
24.00
20.00
16.00
12.00
8.00
4.00
0
8.00
11.30
14.60
17.90
21.20
24.50
27.80
31.10
34.40
37.70
41.00

5.2.4 Livneh and Ishai model (1975)

They proposed that the equilibrium moisture content may be estimated as follows:

$$\frac{EMC}{PL} = 0,86 + 0,14(PL) \quad \ldots\ldots \quad (5.3)$$

where $\frac{EMC}{PL}$ ranges between 0,9 and 1,5.

If this model is applied to local conditions huge errors result. The standard error of estimate is nearly 7. If it is applied between the limits they suggested, however, it is very accurate. The standard error of estimate is only 1,43, but then it is only applicable to 14 per cent of the local cases, which clearly renders it unfit for general use.

5.2.5 Madrid Transport and Soil Mechanics Laboratory model (OECD, 1973)

This investigation reported intervals in which the equilibrium moisture content of subgrade usually fell, in terms of various parameters such as PL, LL and the percentage material finer than 0,075 mm (% - 0,075).

(a) Plastic Limit:

$$0,87(PL) < PEMC < 1,11 \quad \ldots\ldots \quad (5.4)$$

Only 15 per cent of local subgrades fall in this region and 85 per cent outside, as can be seen from Figure 5.4.

(b) Liquid Limit:

$$0,38(LL) < PEMC < 0,49(LL) \quad \ldots\ldots \quad (5.5)$$

Only 29 per cent of local data fall in this region and 71 per cent outside, as can be seen on Figure 5.5.

$$0,33(\% - 0,075) \leqslant PEMC \leqslant 0,60(\% - 0,075) \quad \ldots\ldots \quad (5.6)$$

Only 35 per cent of local data fall in this region and 65 per cent outside, as can be seen in Figure 5.6.

Regression analyses on local data with the above parameters as predictors, resulted in the following:

FIGURE 5.5

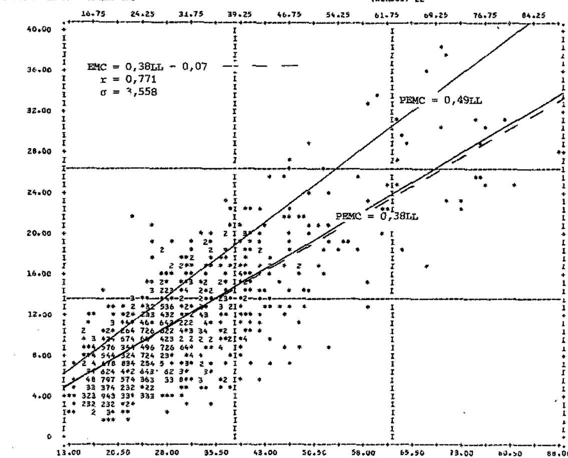
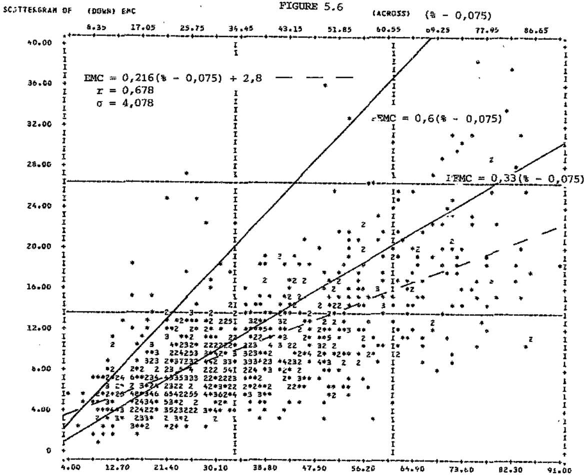

SCATTERGRAM OF (DOWN) EMC
(ACROSS) LL
16.75 24.25 31.75 39.25 46.75 54.25 61.75 69.25 76.75 84.25
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0
13.00 20.50 28.00 35.50 43.00 50.50 58.00 65.50 73.00 80.50 88.00
EMC = 0,38LL - 0,07
r = 0,771
σ = 3,558
PEMC = 0,49LL
PEMC = 0,38LL

FIGURE 5.6


SCATTERGRAM OF (DOWN) EMC (ACROSS) (% - 0,075)
EMC = 0,216(% - 0,075) + 2,8
r = 0,678
σ = 4,078
EMC = 0,6(% - 0,075)
EMC = 0,33(% - 0,075)
8.35 17.05 25.75 34.45 43.15 51.85 60.55 69.25 77.95 86.65
4.00 12.70 21.40 30.10 38.80 47.50 56.20 64.90 73.60 82.30 91.00
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0

(a) Liquid Limit:

$$EMC = 0,38(LL) - 0,07 \quad \ldots\ldots \quad (5.7)$$

The correlation coefficient was 0,771 and the standard error of estimate 3,558 which indicates a good correlation between equilibrium moisture content and LL. The relationship between PL and equilibrium moisture content is reported under Section 5.2.2.

(b) Percentage material finer than 0,075 mm (% - 0,075):

$$EMC = 0,216(\% - 0,075) + 2,795 \quad \ldots\ldots \quad (5.8)$$

The correlation coefficient was 0,678 and the standard error of estimate 4,078 which indicates that there is not a very strong relationship between the percentage material finer than 0,075 mm and equilibrium moisture content.

5.2.6 The USA Navy model (U S NAVY, 1953; OECD, 1973)

This investigation reported an upper limit below which the equilibrium moisture content usually falls:

$$PEMC \leq PL + 2 \quad \ldots\ldots \quad (5.9)$$

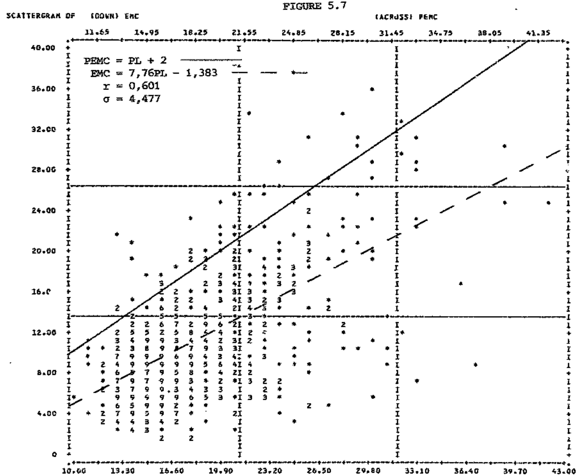
Ninety five per cent of all local subgrade soils falls in this category, as can be seen in Figure 5.7.

This means that this is certainly a rough guide applicable to local conditions, which can be used to give a conservative estimate of equilibrium moisture content.

5.2.7 The Kersten model (1944,1945)

Kersten presented the following limits between which equilibrium moisture content usually falls:

FIGURE 5.7


SCATTERGRAM OF (DOWN) EMC
(ACROSS) PEMC
PEMC = PL + 2
EMC = 7,76PL - 1,383
r = 0,601
σ = 4,477
11.65 14.95 18.25 21.55 24.85 28.15 31.45 34.75 38.05 41.35
10.60 13.30 16.60 19.90 23.20 26.50 29.80 33.10 36.40 39.70 43.00
40.00 36.00 32.00 28.00 24.00 20.00 16.0 12.00 8.00 4.00 0

$0{,}8(PL) < PEMC < 1{,}2(PL)$ .... (5.10)

Only 24 per cent of local subgrade soils falls into this category as can be seen in Figure 5.8.

5.2.8 <u>The Wooltorton model</u> (1958)

This investigation proposed the following relationship:

$PEMC = 1{,}17(PL) - 4$ .... (5.11)

This model has a standard error of estimate of 6,167 when applied to local conditions. The relationship between PL and equilibrium moisture content has been checked and is reported on under Section 5.2.2.

Figure 5.9 shows the scatter resulting when the actual values are plotted against the predicted ones.

5.2.9 <u>Organisation of Economic Co-operation and Development</u> (1973)

This method could not be easily tested for local conditions because comprehensive local data pertaining to this model are not readily available.

5.2.10 <u>The Road Research Laboratory model</u> (RRL, 1977)

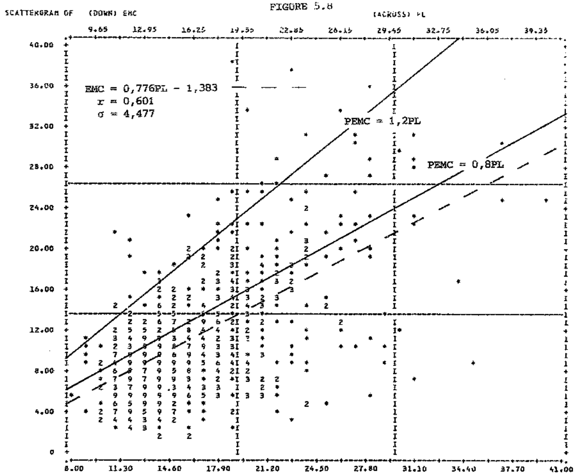
This laboratory also suggested a strong correlation between PL and equilibrium moisture content, which was found not true for local soils. See Section 5.2.2.

5.2.11 <u>Brodie's model</u> (1970)

Brodie developed the following relationship after analysing 330 samples statistically:

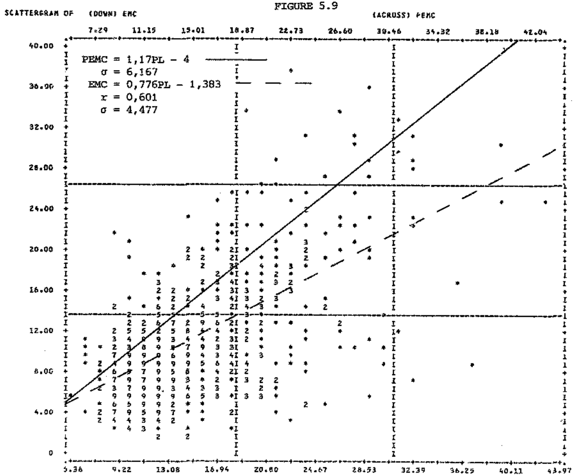
$PEMC = 0{,}825(OMC) - 2{,}7$ .... (5.12)

FIGURE 5.8



SCATTERGRAM OF (DOWN) EMC (ACROSS) PL
EMC = 0,776PL - 1,383
r = 0,601
σ = 4,477
PEMC = 1,2PL
PEMC = 0,8PL
9.65 12.95 16.25 19.55 22.85 26.15 29.45 32.75 36.05 39.35
8.00 11.30 14.60 17.90 21.20 24.50 27.80 31.10 34.40 37.70 41.00
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0

FIGURE 5.9


SCATTERGRAM OF (DOWN) EMC (ACROSS) PEMC
7.29
11.15
15.01
18.87
22.73
26.60
30.46
34.32
38.18
42.04
PEMC = 1,17PL - 4
σ = 6,167
EMC = 0,776PL - 1,383
r = 0,601
σ = 4,477
40.00
36.00
32.00
28.00
24.00
20.00
16.00
12.00
8.00
4.00
0
5.36
9.22
13.08
16.94
20.80
24.67
28.53
32.39
36.25
40.11
43.97

He found a regression coefficient 0,78 when including material from all pavement layers and 0,75 when considering subgrade soils only. This model was tested against the local data and a standard error of estimate of 4,175 was found, but it must be pointed out here that the optimum moisture content for the local data was determined at Proctor compaction while Brodie's samples were compacted at British standard compaction. The change of compaction effort should only result in a relatively small change in regression equation.

When Brodie's predicted values are plotted against the actual values, the scatter in Figure 5.10 results.

It can be seen that the correlation coefficient is significantly higher for local data than that reported by Brodie. The best model using optimum moisture content for local data is:

$$EMC = 1,086(OMC) - 3,39 \quad \ldots\ldots \quad (5.13)$$

Figure 5.11 shows that the modified equation (5.13) fits the local data far better than equation (5.12). The correction that should be applied if the Mod. AASHTO compaction had been used for the determination of the optimum moisture content is developed in Chapter 7, section 7.5.2.2.

It is discouraging to see that a predictor that works so well for local conditions is less applicable to New South Wales conditions. It is possible that the lack of correction for the difference in compactive effort may cause this phenomenon.

5.2.12 Gawith's model (1948)

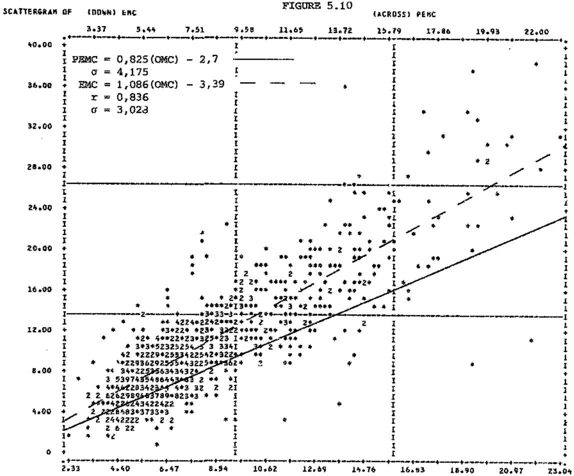
Gawith developed the following two prediction equations for Australian conditions:

$$PEMC = 0,5278(PL) + 0,1183(LL) + 0,2359(\% - 0,075) - 0,0006864(\% - 0,075)^2 + 0,1396(MAR) - 14,29 \quad \ldots\ldots \quad (5.14)$$

$$PEMC = 0,6776(PL) + 0,07875(LL) + 0,2073(\% - 0,075) - 0,000498(\% - 0,075)^2 - 10,681 \quad \ldots\ldots \quad (5.15)$$

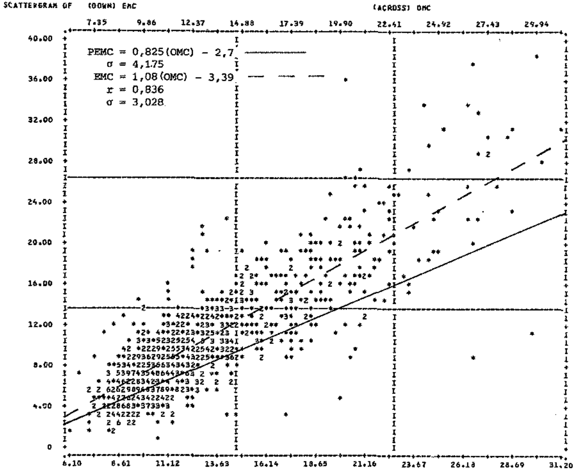
Testing these equations for local conditions, it was found that equation (5.15) is significantly more applicable, despite the extra climatic parameter in equation (5.14). Equation (5.14) had a standard error of estimate of 5,959 and equation (5.15) a standard error of estimate of 3,957.

FIGURE 5.10



SCATTERGRAM OF (DOWN) EMC (ACROSS) PEMC
PEMC = 0,825(OMC) - 2,7
σ = 4,175
EMC = 1,086(OMC) - 3,39
r = 0,836
σ = 3,023
3.37 5.44 7.51 9.58 11.65 13.72 15.79 17.86 19.93 22.00
2.33 4.40 6.47 8.54 10.62 12.69 14.76 16.83 18.90 20.97 23.04
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0

FIGURE 5.11


SCATTERGRAM OF (DOWN) EMC (ACROSS) OMC
PEMC = 0,825(OMC) - 2,7
σ = 4,175
EMC = 1,08(OMC) - 3,39
r = 0,836
σ = 3,028
7.35 9.86 12.37 14.88 17.39 19.90 22.41 24.92 27.43 29.94
6.10 8.61 11.12 13.63 16.14 18.65 21.16 23.67 26.18 28.69 31.20
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0

The best regression equation using above parameters for local conditions is:

$$EMC = 0,019\ (PL) + 0,3065\ (LL) + 0,2163\ (\% - 0,075) - 0,00141\ (\% - 0,075)^2 + 0,198\ (MAR) - 5,13 \quad \ldots\ldots\ldots\ldots \quad (5.16)$$

and

$$EMC = 0,0507\ (PL) + 0,293\ (LL) + 0,2276\ (\% - 0,075) - 0,00148\ (\% - 0,075)^2 - 4,284 \quad \ldots\ldots\ldots\ldots \quad (5.17)$$

Figure 5.12 indicates the scatter of the predicted values of equation (5.14) with the actual values as well as the statistics for the best equation for local conditions with the same parameters. Figure 5.13 shows the same information related to equation (5.15).

From these figures, the expected increase in accuracy with the addition of a climatic variable can be observed, although the increase in accuracy is not large.

5.2.13 Arulunandan's model (1957)

This investigation resulted in the following relationship:

$$PEMC = \frac{PL + (\% - 0,075)}{5} \quad \ldots\ldots\ldots\ldots \quad (5.18)$$

This model was tested for local conditions and a standard error of estimate of 3,828 was found.

The best relationship between above parameters and the equilibrium moisture content for local data is as follows:

$$EMC = 0,166(\% - 0,075) + 0,548\ (PL) - 4 \quad \ldots\ldots\ldots\ldots \quad (5.19)$$

Figure 5.14 shows a plot of the predicted values from equation (5.18) versus the actual values. The statistics of equation (5.19) are also presented on this figure.

5.2.14 Van Der Merwe's model (1967)

An investigation in Rhodesia produced the following relationship:

FIGURE 5.12

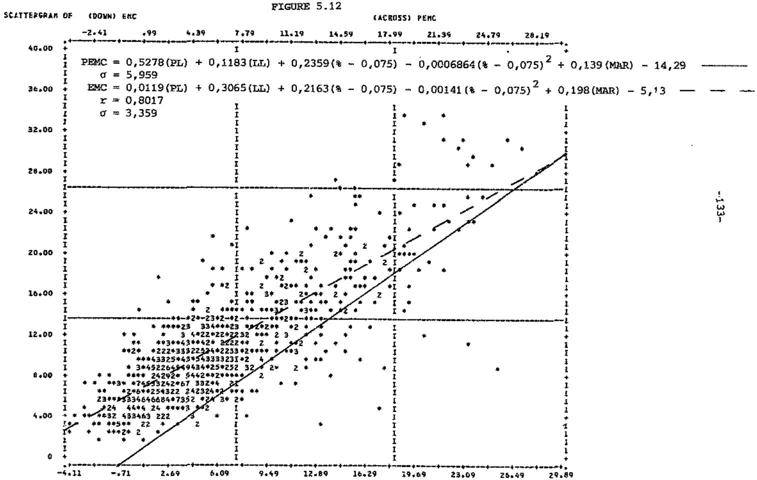

SCATTERGRAM OF (DOWN) EMC (ACROSS) PEMC
-2.41 .99 4.39 7.79 11.19 14.59 17.99 21.39 24.79 28.19
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0
-4.11 -.71 2.69 6.09 9.49 12.89 16.29 19.69 23.09 26.49 29.89
PEMC = 0,5278(PL) + 0,1183(LL) + 0,2359(% - 0,075) - 0,0006864(% - 0,075)² + 0,139(MAR) - 14,29
σ = 5,959
EMC = 0,0119(PL) + 0,3065(LL) + 0,2163(% - 0,075) - 0,00141(% - 0,075)² + 0,198(MAR) - 5,13
r = 0,8017
σ = 3,359

FIGURE 5.13

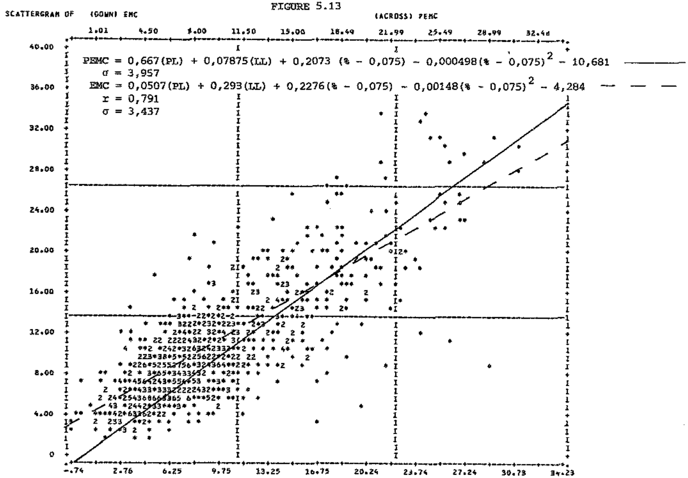

SCATTERGRAM OF (DOWN) EMC
(ACROSS) PEMC
1.01 4.50 8.00 11.50 15.00 18.49 21.99 25.49 28.99 32.48
PEMC = 0,667(PL) + 0,07875(LL) + 0,2073 (% - 0,075) - 0,000498(% - 0,075)^2 - 10,681
σ = 3,957
EMC = 0,0507(PL) + 0,293(LL) + 0,2276(% - 0,075) - 0,00148(% - 0,075)^2 - 4,284
r = 0,791
σ = 3,437
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0
-.74 2.76 6.25 9.75 13.25 16.75 20.24 23.74 27.24 30.73 34.23

FIGURE 5.14

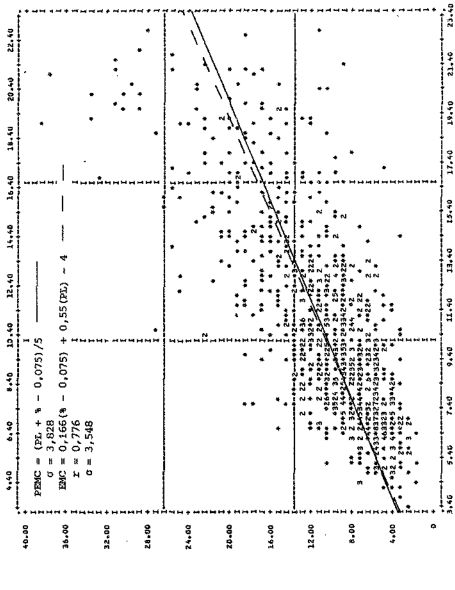
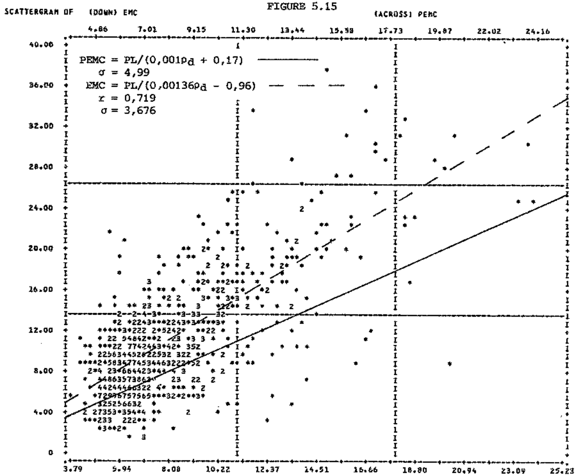

SCATTERGRAM OF (DOWN) EMC (ACROSS) PEMC
PEMC = (PL + % - 0,075)/5
σ = 3,828
EMC = 0,166(% - 0,075) + 0,55(PL) - 4
r = 0,776
σ = 3,548
40.00
36.00
32.00
28.00
24.00
20.00
16.00
12.00
8.00
4.00
0
3.40
4.40
5.40
6.40
7.40
8.40
9.40
10.40
11.40
12.40
13.40
14.40
15.40
16.40
17.40
18.40
19.40
20.40
21.40
22.40
23.40

FIGURE 5.15


SCATTERGRAM OF (DOWN) EMC
(ACROSS) PEMC
PEMC = PL/(0,001ρd + 0,17)
σ = 4,99
EMC = PL/(0,00136ρd - 0,96)
r = 0,719
σ = 3,676
4.86 7.01 9.15 11.30 13.44 15.58 17.73 19.87 22.02 24.16
3.79 5.94 8.08 10.22 12.37 14.51 16.66 18.80 20.94 23.09 25.23
40.00 36.00 32.00 28.00 24.00 20.00 16.00 12.00 8.00 4.00 0

$$PEMC = \frac{PL}{(0{,}001(\rho_d) + 0{,}17)} \quad \ldots\ldots\ldots\ldots \quad (5.20)$$

This model has a standard error of estimate of 4,990 when applied to local conditions. Figure 5.15 gives an indication of how this model performed against the actual values. The best regression formula for local conditions using these parameters is also shown.

In order to improve or optimise this relationship for local subgrades, the following coefficients were obtained:

$$EMC = \frac{PL}{0{,}00136(MDD) - 0{,}96} \quad \ldots\ldots\ldots\ldots \quad (5.21)$$

Figure 5.15 shows that this equation fits local data better. Or alternatively, using the same predictors:

$$EMC = 0{,}288(PL) - 0{,}0232\ (MDD) + 50 \quad \ldots\ldots\ldots\ldots \quad (5.22)$$

5.2.15 Summary

Before summarizing all the available empirical methods and ranking them according to accuracy, it is interesting to note that less five per cent of all subgrade soils in Transvaal ever reach saturation. This is not an accurate figure, because the saturated moisture contents were calculated assuming the specific gravity to be constant at 2,65. It nevertheless indicates how conservative the design assumption has been to date.

Table 5.1 summarizes all the tested available models and the modifications to suit local conditions best.

Table 5.2 gives the various tested models in order of accuracy. From this table it is clear that, of all the parameter combinations so far suggested by researchers, a simple optimum moisture content function is the most accurate predictor of the equilibrium moisture content.

Although a correlation coefficient of 0,836 will be acceptable for most engineering applications, it is not the case in this instance because the standard error of estimate is relatively high. If the Brodie model is used for example, and predicts the equilibrium moisture content to be 20 per cent, it means that there is a 15 per cent chance that the actual moisture content is larger than 20 % x 1,38 = 27,6 % and the strength properties of problem subgrades will vary significantly in this interval.

TABLE 5.1: SUMMARY OF EXISTING MOISTURE PREDICTION TECHNIQUES

Author	Predicted equilibrium moisture content (a) Original model of author indicated (b) Modified model developed locally	Local values in this range %	Single or multiple correlation coefficient r or R	Error at 95 % level when applied to local conditions*** %
Swanberg and Hansen (1946)	(a) 1,16(PL) - 7,4		-	44
	(b) 0,78(PL) - 1,4		0,60	41
Swanberg and Hansen* (1946)	(a) 1,16(PL) - 7,4		-	43
	(b) 0,88(PL) - 3,4		0,50	40
Haliburton (1971)	(a) EMC $\leq$ 1,1(PL) to 1,3(PL)	95	-	
Livneh and Ishai**(1975)	(a) $0{,}86(PL) + 0{,}14(PL)^2$		-	62
	(b) $1{,}42(PL) - 0{,}0088(PL)^2 - 3{,}2$		0,92	13
Madrid Transport and Soil Mechanics Laboratory (OECD) (1973)	(a) 0,87(PL) < EMC < 1,11(PL)	15	-	
	(b) 0,78(PL) - 1,34		0,60	41
	(a) 0,38(LL) < EMC < 0,49(LL)	29	-	
	(b) 0,38(LL) - 1		0,77	32
	(a) 0,33(% - 0,075) < EMC < 0,60(% - 0,075)	35	-	
	(b) 0,22(% - 0,075) + 2,8		0,67	37
USA Navy (1953)	(a) EMC $\leq$ PL + 2	95	-	
	(b) 0,78(PL) - 1,4		0,60	41
Kersten (1945)	(a) 0,8 (PL) < EMC < 1,2(PL)	24	-	
	(b) 0,78(PL) - 1,4		0,60	41
Woolterton (1958)	(a) 1,17(PL) - 4		-	56
	(b) 0,78(PL) - 1,4		0,60	41
Brodie (1970)	(a) 0,825(OMC) - 2,7		-	38
	(b) 1,1(OMC) - 3,4		0,94	28
Gawith (1948)	(a) $0{,}5278(PL) + 0{,}1183(LL) + 0{,}2359(\% - 0{,}075) - 0{,}0006864(\% - 0{,}075)^2 + 0{,}1306(MAR) - 14{,}29$		-	54
	(b) $0{,}0119(PL) + 0{,}3065(LL) + 0{,}2163(\% - 0{,}075) - 0{,}00141(\% - 0{,}075)^2 + 0{,}198(MAR) - 5{,}13$		0,80	31
	(a) $0{,}6776(PL) + 0{,}07875(LL) + 0{,}2073(\% - 0{,}075) - 0{,}000498(\% - 0{,}075)^2 - 10{,}691$		-	36
	(b) $0{,}0507(PL) + 0{,}293(LL) + 0{,}2276(\% - 0{,}075) - 0{,}00148(\% - 0{,}075)^2 - 4{,}28$		0,79	31
Arulanandan (1957)	(a) $\frac{PL + (\% - 0{,}075)}{5}$		-	35
	(b) 0,17(% - 0,075) + 0,55(PL) - 4		0,77	32

TABLE [illegible].1: (Continued)

Author	Predicted equilibrium moisture content (a) *Original model of author indicated* (b) Modified model developed locally	Local values in this range %	Single or multiple correlation coefficient r or R	Error at 85 % level when applied to local conditions*** %
Van Der Merwe (1967)	(a) $\frac{PL}{(0,001(MDD) + 0,17)}$		-	45
	(b) $\frac{PL}{(0,00136(MDD) - 0,96)}$ or $0,23(PL) - 0,0232(MDD) + 50$		0,82	33 or 29

Notes: * Only between limits $15 < PL < 30$

** Only between limits $0,9(PL) \leq EMC \leq 1,5(PL)$ (Only 14 per cent of local data fall in this range.)

*** There is a 15 per cent chance that the actual moisture content will exceed the predicted moisture content increased by the percentage error

TABLE 5.2: ORDER OF ACCURACY OF EXISTING MODELS

Name	Parameters used	Multiple correlation coefficient of modified model	Standard error of estimate (a) Original model (b) Modified model		
Brodie	OMC	0,83604	(a)	4,175	(38* %)
		0,83604	(b)	3,02815	(28 %)
Gawith	PL, LL, MAR, (%-0,075)		(a)	5,959	(54 %)
		0,80177	(b)	3,35902	(31 %)
Gawith	PL, LL, (%-0,075)		(a)	3,957	(36 %)
		0,79128	(b)	3,43675	(31 %)
[illegible]rid	LL		(a)	-	
		0,77147	(b)	3,55859	(32 %)
Arulanandan	PL, (%-0,075)		(a)	3,828	(35 %)
		0,76782	(b)	3,60089	(32 %)
Van Der Merwe	PL, MDD		(a)	4,990	(45 %)
		0,76304	(b)	3,16795	(33 %)
Madrid	(%-0,075)		(a)	-	
		0,67809	(b)	4,07765	(37 %)
Swanberg and Hansen	PL		(a)	4,804	(44 %)
		0,60064	(b)	4,47746	(41 %)

* The prediction accuracies at the 85 per cent level are given in brackets.

Because of the inadequacy of the above models for the purpose in mind, it was decided to include other soil parameters in the local data in an attempt to improve the accuracy of the above models. It was felt that if the new regression analysis on the comprehensive data base does not yield satisfactory answers, the more theoretical techniques would be persued.

It can be concluded that although these models are not sufficiently accurate to be incorporated into a sophisticated design procedure, they are very valuable for use under circumstances where the known error involved is acceptable.

5.3 General local trends

It was concluded in Chapter 3 that many conflicting observations about the subgrade moisture regime have been reported in the literature, and it is mainly for this reason that a comprehensive analysis of existing data was carried out in order to establish trends applicable to local conditions. This was done by breaking the criterion (moisture content) down by various different parameters in turn. Trends or the absence of trends could be detected in this way by examining the means and variances of the criterion among the various subgroups. Tables showing this information are presented in the text, while figures depicting the variation of moisture content with the various parameters are shown in Appendix C. This analysis will also serve as a useful source of reference.

A brief summary of the descriptive statistics used to establish these trends is given below. Further details may be found in most standard texts on statistics (Blalock, 1972; Downie and Heath, 1970; Ostle, 1969).

(a) Levels of measurement

The level of measurement of a variable indicates the ordering and distance properties inherent to the measurement of that variable.

(i) Nominal-level measurement

Here no assumption is made about the values being assigned to the data. Each value is a distinct category and the value itself serves merely as a name, for example the city where a person was born.

(ii) Ordinal-level measurements

When it is possible to rank order all the categories according to some criterion, then the ordinal level of measurement has been achieved, for example the classification of surface drainage in categories good, fair and poor.

(iii) Interval-level measurements

In addition to ordering, the interval level of measurement has the property that the distance between the categories are defined in terms of fixed and equal units, for example the measurement of temperature in degrees C or degrees F. The important thing to note is that the interval scale does not have an inherently determined zero point. Zero ^{o}C does not imply the absence of heat.

(iv) Ratio-level measurements

The ratio level of measurement has all the properties of an interval scale with the additional *property* that the zero point is inherently defined by the measurement system, for example the measurement of length.

(v) Special case of dichotomics

A dichotomy is a variable with only two possible categories, *such as sex*. Table 5.3 shows the level of measurement of variables used in this analysis.

(b) Minimum, maximum and range

The minimum and maximum, of course, denote the smallest and largest value of variable encountered among all cases, while the range is the minimum subtracted from the maximum. These are suitable for use with variables measured at any level.

(c) Mode

The mode is the variable that occurs most often and can be used with variables measured at any level.

(d) *Median*

The median is the numerical value of the middle case or the case lying exactly on the fiftieth percentile, once all the cases have been ordered from highest to lowest.

TABLE 5.3: LEVEL OF MEASUREMENT OF VARIOUS PARAMETERS

Parameter	Abbreviation	Level of measurement
Elevation	Elev.	Nominal
Surface drainage	Surdran.	Ordinal
Road condition	Rcon.	Ordinal
Crack condition	Crcon.	Dichotomics
Type crack 1 (crocodile)	Typcr 1	Nominal
Type crack 2 (block)	Typcr 2	Nominal
Type crack 3 (longitudinal)	Typcr 3	Nominal
Type crack 4 (transverse)	Typcr 4	Nominal
Rut condition	Rutcon.	Dichotomics
Thickness of bitumen surface	Bitsur.	Ratio
Thickness of total cover	Totcover	Ratio
Equilibrium moisture content	EMC	Ratio
Grading modulus	GM	Ratio
Liquid limit	LL	Ratio
Plasticity index	PI	Ratio
Linear shrinkage	LS	Ratio
Sand equivalent	SE	Ratio
Maximum density	MDD	Ratio
Optimum moisture content	OMC	Ratio
Maximum CBR	CBR	Ratio
Percentage material passing 0,425	% - 0,425	Ratio
Percentage material passing 0,075	% - 0,075	Ratio
Mean annual rainfall	MAR	Ratio
Climatic N-value	N	Interval
Climatic I-value	I	Interval

(e) Mean

The mean is the most common measurement of central tendency for variables measured at the interval level. It is calculated as:

$$\bar{X} = \frac{\Sigma_{i=1}^{N} (x_i)}{N}$$

where x_i equals the score of each case and N represents the total number of valid cases.

(f) Variance

Variance, denoted by S^2, is a measurement of the dispersion of the data about the mean of an interval level variable. Mathematically, it is the average squared deviation from the mean.

$$S^2 = \frac{\Sigma_{i=1}^{N} (x_i - \bar{X})^2}{N - 1}$$

Squaring the deviations from the mean takes account of all differences from the mean, including negative differences, and it gives additional weight to the extreme cases. It is a very important statistic, as one of the main aims of research is to "explain" variance.

(g) Standard deviation

The standard deviation is another measure of the dispersion about the mean of an interval level variable. It is simply the square root of the variance. The advantage of the standard deviation is that it has a more intuitive interpretation, being of the same units as the original variable.

(h) Standard error

If one were to draw an infinite number of equal-sized samples from a given population, the mean of each sample would be an estimate of the true population mean, but not all of them would be identical. The pattern of these means would actually constitute a normal distribution and would have a standard deviation. The standard deviation of this distribution is the standard error. Thus, the standard error helps us to determine the potential degree of discrepancy between the sample mean and the unknown population mean. It cannot be computed exactly, but it can be estimated by dividing the standard deviation by the square root of the number of cases. It requires interval level measurements and different kinds of standard errors are used, e.g. standard error of mean, standard error of estimate, etc. The latter gives an indication of the prediction accuracy of the regression equation. r^2 gives the amount of variation explained and $(1-r^2)$ the unexplained variation. The standard error of estimate is simply the standard deviation of the actual criterion values around the predicted criterion values.

(i) Skewness

Skewness is the statistic needed to determine the degree to which a distribution of cases approximates a normal curve, since it measures deviations from symmetry. It will take a value of zero when the distribution is a completely symmetric bell-shaped curve. A positive value indicates that the cases are clustered more to the left of the mean with most of the extreme values to the right and vice versa. It requires interval level measurements.

(j) Kurtosis

Kurtosis is a measure of the relative peakedness or flatness of the curve defined by the distribution of cases. A normal distribution will have a kurtosis of zero. If the kurtosis is positive, then the distribution is more peaked than a true normal distribution and vice versa. It requires interval level data.

5.3.1 Elevation

Table 5.4 shows the equilibrium moisture content broken down by elevation. It can be seen from this table that the elevation of the road surface above the surrounding topography does not influence the equilibrium moisture regime markedly. On the basis of a knowledge about the topography of the site alone, one can in general say nothing about the moisture regime in the subgrade. The only exception is that sidefills appear to be drier than average. A reason for this apparent insignificance of elevation may be the fact that a fill situation does not necessarily imply good drainage, which means that relatively wet layers near the top of the fill may exist. The fact that different subgrade materials will be encountered in cut, fill and flat areas is also not taken into account.

TABLE 5.4: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN BY ELEVATION

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Elev	1	Flat	10,5954	5,6978	32,4648	(778)
Elev	2	Fill	10,9177	5,1444	26,4649	(130)
Elev	3	Side fill	9,0602	4,2711	18,2420	(95)
Elev	4	Cut	10,6163	4,9893	24,8931	(49)
Elev	5	Vlei	6,9000	0	0	(1)
Elev	12	Flat fill	12,5000	8,5418	72,9620	(11)
Elev	13	Flat side fill	7,0500	2,8758	8,2700	(4)
Elev	15	Flat vlei	19,6000	0	0	(1)
Elev	25	Fill vlei	10,2000	3,1113	9,6800	(2)
Elev	34	Side fill cut	3,1000	0	0	(1)

5.3.2 Surface Drainage

From Table 5.5 it may be seen that although the quality of the surface drainage is not strongly related to the subgrade equilibrium moisture content, there is a significant trend for the moisture content to increase as the surface drainage provided gets worse. It is not at all surprising that the quality of surface drainage does not correlate better with the subgrade moisture conditions, for the following reasons:

(a) It has been found (Scala, 1967) that the evaluation of drainage facilities, even by a team of experienced engineers, is extremely difficult.

(b) The influence of sub-surface drains which are known to be effective in removing excess water from the pavement layers (Cedergren, 1973) is not included in this assessment.

It can, however, on the basis of this information, be stated with confidence that the provision of good surface drainage facilities will assist in the improvement of the subgrade moisture regime. This means that provision must be made for the water to run off the pavement and away into the field. It should <u>not</u> be allowed to pond on the pavement or next to it.

TABLE 5.5: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY SURFACE DRAINAGE

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Surdran	1	Good	10,1595	5,2252	27,3025	(236)
Surdran	2	Fair	10,3373	5,3530	28,6543	(520)
Surdran	3	Poor	11,0320	6,0064	36,0771	(316)

5.1.3 Road Condition

In order to classify the condition of the road, the investigators scrutinized the immediate vicinity of the borehole. If the area was potholed and patched, the road was classified as failed. This method of random sampling is usually employed with success, but in this case biased sampling should have been done, i.e. the exact spot where the hole was to be drilled should have been examined, because the pavement might be failed in one spot and be unfailed at a spot two meters away. This means that some of the sites drilled in "failed" pavements could in fact have been in unfailed sections. Another difficulty which arises with an investigation like this is that it is extremely difficult to ascribe failure to the subgrade specifically. In this case the road as a whole was judged to be either failed or not. No information (except rut depth) is available to ascertain the contribution of the subgrade to the failure observed on the surface.

From the information presented in Table 5.6 it can, however, be seen that the equilibrium moisture content in the subgrade increases as the pavement fails. This suggests that the method of wetting up of a subgrade

TABLE 5.6: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY ROAD CONDITION

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Rcon	1	Not failed	9,9362	5,2231	27,2805	(784)
Rcon	2	Border line	11,3177	5,4845	30,0800	(124)
Rcon	3	Failed	12,5963	6,3965	40,9151	(164)

in Transvaal under these conditions is generally through the pavement layers, from the atmosphere, and not from the groundwater. This is in conflict with the observation that drainage does not affect the equilibrium moisture content significantly, because if equilibrium moisture content was dependent on the atmosphere, drainage would in fact have a signifcant effect on it. This apparent incongruity can be explained by the fact that the trends presented here are very general and that sites where all kinds of mechanisms of wetting up are present, are represented. A warning should therefore be sounded that these trends should not be extended to other areas without careful consideration.

5.3.4 Crack Condition

This parameter shows no significant relation with equilibrium moisture content. It does not influence the equilibrium moisture content of the subgrade. This may be due to the fact that some of the sites wetted up from the groundwater and some from the atmosphere. These are two accepted mechanisms of wetting up, they both occur in the Transvaal and they therefore obscure the influence of the parameters discussed so far because they normally cause opposite trends. Another factor that will depress the influence of the factors discussed so far, is that some sites were sampled in the dry and some in the wet season. What is significant, however, from this analysis, is that although the mean equilibrium moisture content for the uncracked condition is slightly higher than that of the cracked condition, the variance of the cracked condition is higher, which means that a more stable equilibrium moisture content is reached under an uncracked pavement. It should also be remembered that if the surface is cracked, it does not necessarily mean that the base and subbase are also cracked. It is possible that a cracked surface does not increase the pavement permeability significantly as far as the subgrade is concerned. Cedergren (1973) found that the base and subbase moisture content is sensitive to a cracked pavement. For this whole analysis two assumptions have been made that are probably not strictly true for the Transvaal. Firstly it was assumed that an equilibrium moisture content existed and that it was therefore immaterial whether the sampling was done in winter or in summer. Secondly it was implicitly assumed that the same method of wetting up existed at all sites. Further work which would involve measuring the depth to the water table should clarify exactly what the situation is. Table 5.7 shows how the moisture content varies as the road condition deteriorates.

TABLE 5.7: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY CRACK CONDITION

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Crcon	1	Not cracked	10,7526	5,3141	28,2398	(630)
Crcon	2	Cracked	10,1471	5,8174	33,8423	(442)

5.3.5 Rut Condition

It can be seen from Table 5.8 that although the cracked pavements have lower equilibrium moisture contents than the uncracked ones, the rutted pavements have a higher equilibrium moisture content than those not rutted. The relation between surface rutting and subgrade moisture content is also highly significant. This may be due to the fact that rutting is usually caused by the subgrade distress and thus, if the subgrade is weaker (wetter), one will expect more rutting. Cracks in the pavement are usually due to the distress of the structural pavement layers and are less dependent on the subgrade equilibrium moisture content. It can thus with some confidence be said that one of the most significant visual surface indicators of subgrade wetting is rutting.

TABLE 5.8: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY RUT CONDITION

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Rutcon	1	Not rutted	10,0707	5,4137	29,3081	(782)
Rutcon	2	Rutted	11,6686	5,6888	32,3623	(290)

5.3.6 Thickness of bitumen surfacing

A general tendency of equilibrium moisture content to increase as the thickness of bitumen surfacing increases is noted in Table 5.9. The variation of equilibrium moisture content under thin pavements is considerably higher than the variation under thicker pavements. This indicates that a less permeable surfacing results in more stable moisture conditions. It must be pointed out here that the fact that the subgrade under thicker pavements is wetter than the subgrade under thinner pavements, suggests that the wetting up mechanism is from the groundwater and not from the atmosphere, or else it may be due to the fact that thicker pavements are required over weaker subgrades (higher plasticity), which would result in higher equilibrium moisture content. It must be concluded that both methods of wetting up are present and it remains to be seen which occurs where and how the above parameter influences each case.

TABLE 5.9: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY THICKNESS OF BITUMEN SURFACING

Variable	Code	Va[illegible] label (mm)	Mean	Std. dev.	Variance	N
Fo[illegible] entire population			10,5029	5,5326	30,6093	(1072)
Bitsur	2	From 5,0[illegible] to 10	7,5500	6,1518	37,8450	(2)
Bitsur	3	From [illegible] to 15	8,5824	3,3690	11,3503	(17)
Bitsur	4	From 15,01 to 20	10,5413	5,0125	25,1251	(201)
Bitsur	5	From 20,01 to 25	9,3743	4,9778	24,7780	(113)
Bitsur	6	From 25,01 to 30	10,3688	5,5170	30,4372	(349)
Bitsur	7	From 30,01 to 35	11,6545	5,9999	35,9985	(55)
Bitsur	8	From 35,01 to 40	10,9240	5,8746	34,5111	(150)
Bitsur	9	From 40,01 to 45	10,4074	6,9421	48,1930	(27)
Bitsur	10	From 45,01 to 50	10,4949	5,7082	[illegible]2,5839	(69)
Bitsur	12	From 55,01 to 60	10,0053	3,5614	12,6839	(19)
Bitsur	13	From 60,01 to 6[illegible]	20 8000	2,4042	5,7800	(2)
Bitsur	14	From 65,01 to 70	11,6500	4,9130	24,1374	(20)
Bitsur	15	From 70,01 to 75	9,200[illegible]	0	0	(1)
Bitsur	16	From 75,01 to 80	10,2615	4,8418	23,4433	(26)
Bitsur	18	From 85,01 to 90	12,4091	8,2685	68,3689	(11)
Bitsur	19	From 90,01 to 95	6,0500	0,4950	0,2450	(2)
Bitsur	20	From 95,01 to 100	15,7333	12,3447	152,3907	(6)
Bitsur	22	From 105,01 to 110	16,6000	0	0	(1)
Bitsur	24	From 115,01 to 120	16,3000	0	0	(1)

5.3.7 Thickness of total cover

There is a very significant trend for the equilibrium moisture content to increase as the thickness of total cover over the subgrade increases, as can be seen in Table 5.10. This is undoubtedly due to the necessity for thicker pavements over weaker subgrades.

TABLE 5.10: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY THICKNESS OF TOTAL COVER

Variable	Code	Value label (mm)	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Totcover	1	From 0 to 100	6,4000	3,1528	9,9400	(5)
Totcover	2	From 101 to 200	8,6711	4,8246	23,2770	(96)
Totcover	3	From 201 to 300	9,2171	4,1244	17,0107	(228)
Totcover	4	From 301 to 400	9,9683	4,4074	19,4249	(258)
Totcover	5	From 401 to 500	10,6496	5,3929	29,0831	(212)
Totcover	6	From 501 to 600	11,5879	5,8005	33,6456	(141)
Totcover	7	From 601 to 700	13,3368	7,4635	55,7039	(68)
Totcover	8	From 701 to 800	13,7105	8,5163	72,5280	(38)
Totcover	9	From 801 to 900	16,2000	8,3674	70,0129	(18)
Totcover	10	From 901 to 1000	19,9500	9,4747	89,7700	(4)
Totcover	11	From 1001 to 1100	6,9000	3,8184	14,5800	(2)
Totcover	13	From 1101 to 1300	8,5000	0	0	(1)
Totcover	15	From 1301 to 1500	8,2000	0	0	(1)

5.3.8 Grading modulus

From the data presented it is clear that as the grading modulus decreases, so the equilibrium moisture content increases (Table 5.11). This means that the coarser the material gets, the lower the equilibrium moisture content becomes. The relationship is not linear and it has to be transformed before it can be used in any linear regression analysis. This is to be expected because, as the material gets finer, so its ability to hold water increases with some power related to the particle surface area.

TABLE 5.11: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY GRADING MODULUS

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Gradmod	1	From 0 to 0,3	18,0568	9,5332	90,8810	(25)
Gradmod	2	From 0,31 to 0,6	16,8693	6,7243	45,2161	(114)
Gradmod	3	From 0,61 to 0,9	12,0839	4,9252	24,2578	(223)
Gradmod	4	From 0,91 to 1,2	9,4047	4,5691	20,8763	(214)
Gradmod	5	From 1,21 to 1,5	9,3358	3,8415	14,7572	(137)
Gradmod	6	From 1,51 to 1,8	8,5502	3,2999	10,8894	(140)
Gradmod	7	From 1,81 to 2,1	8,3350	4,0246	16,1977	(140)
Gradmod	8	From 2,11 to 2,4	6,8663	2,6577	7,0631	(65)
Gradmod	9	From 2,41 to 2,7	6,2929	2,5901	6,7084	(14)

5.3.9 Liquid limit

This parameter has a strong linear relationship with equilibrium moisture content and as liquid limit increases, so the equilibrium moisture content increases linearly (Table 5.12). This is not a surprising observation, although the high correlation between equilibrium moisture content and liquid limit is not apparent from the literature (correlation coefficient, $r = 0{,}77$). It is also interesting to note that the equilibrium moisture content never reached the liquid limit in any of the approximately 1 200 samples tested.

5.3.10 Plastic limit

The plastic limit also has a very significant relation with equilibrium moisture content. As the plastic limit increases, so the equilibrium moisture content increases linearly (Table 5.13). Only about five per cent of local subgrades ever wet up to their plastic limits.

TABLE 5.12: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY LIQUID LIMIT

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30.6093	(1072)
LL	1	From 0 to 5	6,7811	3,0246	9,1481	(102)
LL	3	From 10.C. to 15	6,1450	2,7871	7,7679	(20)
LL	4	From 15,01 to 20	6,9694	2,3639	5,5879	(206)
LL	5	From 20,01 to 25	8,7305	2,7965	7,8204	(226)
LL	6	From 25,01 to 30	10,6266	3,5625	12,6915	(204)
LL	7	From 30,0. to 35	12,0814	4,0946	16,7660	(118)
LL	8	From 35,01 to 40	13,9159	4,4842	20,1078	(82)
LL	9	From 40,01 to 45	15,6500	4,4345	19,6647	(38)
LL	10	From 45,01 to 50	18,4552	5,7452	33,0068	(29)
LL	11	From 50,01 to 55	19,4818	4,4759	20,0336	(11)
LL	12	From 55,01 to 60	23,0750	7,5120	56,4307	(8)
LL	13	From 60,01 to 65	25,3900	4,3460	18,8877	(10)
LL	14	From 65,01 to 70	31,5000	8,6073	74,0850	(5)
LL	15	From 70,01 to 75	27,7000	5,9672	35,6080	(6)
LL	16	From 75,01 to 80	28,6800	2,8438	8,0870	(5)
LL	17	From 80,01 to 85	25,000	0	0	(1)
LL	18	From 85,01 to 90	28,5000	0	0	(1)

TABLE 5.13: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY PLASTIC LIMIT

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,9020	5,5973	31,3297	(967)
PL	1	From 0 to 12	7,9425	3,2641	10,6541	(186)
PL	2	From 12,1 to 15	8,8631	3,2814	10,7678	(360)
PL	3	From 15,1 to 17	10,9979	3,9939	15,9511	(142)
PL	4	From 17,1 to 22	13,8277	5,6624	32,0632	(196)
PL	5	From 22,1 to 42	19,3048	8,0101	64,1612	(83)

TABLE 5.12: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY LIQUID LIMIT

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
LL	1	From 0 to 5	6,7811	3,0246	9,1481	(102)
LL	3	From 10.01 to 15	6,1450	2,7871	7,7679	(20)
LL	4	From 15,01 to 20	6,9694	2,3639	5,5879	(206)
LL	5	From 20,01 to 25	8,7305	2,7965	7,8204	(226)
LL	6	From 25,01 to 30	10,6266	3,5625	12,6915	(204)
LL	7	From 30,01 to 35	12,0814	4,0946	16,7660	(118)
LL	8	From 35,01 to 40	13,9159	4,4842	20,1078	(82)
LL	9	From 40,01 to 45	15,6500	4,4345	19,6647	(38)
LL	10	From 45,01 to 50	18,4552	5,7452	33,0068	(29)
LL	11	From 50,01 to 55	19,4818	4,4759	20,0336	(11)
LL	12	From 55,01 to 60	23,0750	7,5120	56,4307	(8)
LL	13	From 60,01 to 65	25,3900	4,3460	18,8877	(10)
LL	14	From 65,01 to 70	31,5000	8,6073	74,0850	(5)
LL	15	From 70,01 to 75	27,7000	5,9672	35,6080	(6)
LL	16	From 75,01 to 80	28,6800	2,8438	8,0870	(5)
LL	17	From 80,01 to 85	25,000	0	0	(1)
LL	18	From 85,01 to 90	28,5000	0	0	(1)

TABLE 5.13: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY PLASTIC LIMIT

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,9020	5,5973	31,3297	(967)
PL	1	From 0 to 12	7,9425	3,2641	10,6541	(186)
PL	2	From 12,1 to 15	8,8631	3,2814	10,7678	(360)
PL	3	From 15,1 to 17	10,9979	3,9939	15,9511	(142)
PL	4	From 17,1 to 22	13,8277	5,6624	32,0632	(196)
PI	5	From 22,1 to 42	19,3048	8,0101	64,1612	(83)

5.3.11 Plasticity index

The plasticity index also has a significant linear relationship with equilibrium moisture content and as the plasticity index increases, so the equilibrium moisture content increases.

It is interesting to note that of all the Atterberg Limits, liquid limit shows the best relationship with the equilibrium moisture content, as the table of correlation coefficients r below indicates.

	EMC
LL	0,77
PI	0,73
PL	0,60

Of all the Atterberg Limits, plastic limit correlates worst of all with the moisture content. This is surprising in the light of the fact that of all the existing models developed to predict moisture conditions, the majority use plastic limit as the most important predictor.

TABLE 5.14: DESCRIPITON OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY PLASTICITY INDEX

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
PI	1	From 0 to 5	6,9089	2,8428	80,813	(238)
PI	2	From 5,01 to 10	8,4103	3,2141	10,3302	(291)
PI	3	From 10,01 to 15	10,6284	3,7551	14,1007	(293)
PI	4	From 15,01 to 20	12,6366	4,1926	17,5780	(134)
PI	5	From 20,01 to 25	16,4113	4,4760	20,0349	(53)
PI	6	From 25,01 to 30	20,5826	5,8195	33,8660	(23)
PI	7	From 30,01 to 35	20,7813	4,7715	22,7670	(16)
PI	8	From 35,01 to 40	28,7125	5,3448	28,5670	(8)
PI	9	From 40,01 to 45	25,9429	5,0395	25,3962	(7)
PI	10	From 45,01 to 50	29,6600	5,7518	33,0830	(5)
PI	11	From 50,01 to 55	32,0667	6,6124	43,7233	(3)
PI	12	From 55,01 to 60	28,5000	0	0	(1)

5.3.12 Bar linear shrinkage

Bar linear shrinkage also has a strong linear relationship with equilibrium moisture content and as linear shrinkage increases, so equilibrium moisture content increases (Table 5.15).

TABLE 5.15: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY LINEAR SHRINKAGE

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
LS	1	From 0 to 5	7,8646	3,0959	9,5846	(570)
LS	2	From 5,01 to 10	11,7177	4,2817	18,3329	(408)
LS	3	From 10,01 to 15	18,9300	5,6195	31,5789	(70)
LS	4	From 15,01 to 20	26,6650	5,5557	30,8656	(20)
LS	5	From 20,01 to 25	34,2750	5,0507	25,5092	(4)

5.3.13 Sand equivalent

This parameter has a definite relationship with equilibrium moisture content. As sand equivalent decreases, so equilibrium moisture content increases (Table 5.16). This is not a linear relationship and this parameter will have to be transformed before it can be used in any regression analysis.

5.3.14 Maximum density

This parameter is inversely proportional (tends to a hyperbola) to equilibrium moisture content (Table 5.17) for the range of densities considered.

TABLE 5.16: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY SAND EQUIVALENT

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
SE	1	From 0 to 5	16,3231	7,8569	61,7307	(135)
SE	2	From 5,01 to 10	13,5914	5,0792	25,7978	(187)
SE	3	From 10,01 to 15	10,5965	3,9177	15,3480	(310)
SE	4	From 15,01 to 20	7,9077	3,0090	9,0542	(273)
SE	5	From 20,01 to 25	6,5509	2,9838	8,9032	(126)
SE	6	[illegible] to 30	6,1333	2,8272	7,9933	(30)
SE	7	[illegible] to 35	6,2375	4,2614	18,1598	(8)
SE	9	From [illegible] to 45	4,8500	0,2121	0,0450	(2)
SE	10	From 45,01 to 50	1,2000	0	0	(1)

TABLE 5.17: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY MAXIMUM DENSITY

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Maxden	1	From 0 to 1500	22,2005	10,4115	108,3989	(40)
Maxden	2	From 1501 to 1650	19,2829	4,5021	20,2686	(82)
Maxden	3	From 1651 to 1700	14,6306	4,1717	17,4034	(49)
Maxden	4	From 1701 to 1750	13,8462	2,8598	8,1783	(39)
Maxden	5	From 1751 to 1800	13,1940	3,5420	12,5454	(67)
Maxden	6	From 1801 to 1850	11,5128	3,4643	12,0014	(86)
Maxden	7	From 1851 to 1900	10,1450	3,0466	9,2819	(100)
Maxden	8	From 1901 to 1950	9,1512	2,5555	6,5304	(151)
Maxden	9	From 1951 to 2000	7,7745	2,5792	6,6524	(196)
Maxden	10	From 2001 to 2100	6,7122	2,4081	5,7988	(206)
Maxden	11	From 2101 to 2250	6,3827	2,0540	4,2191	(52)
Maxden	12	From 2250 to 2400	6,0500	1,9157	3,6700	(4)

5.3.15 *Optimum moisture content*

This parameter is directly proportional to equilibrium moisture content and a strong relationship exists (Table 5.18).

TABLE 5.18: *DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY OPTIMUM MOISTURE CONTENT*

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
OMC	1	From 0 to 10	6,7122	2,7117	7,3533	(313)
OMC	2	From 10,1 to 12	8,3402	2,5090	6,2948	(296)
OMC	3	From 12,1 to 15	11,2459	3,2436	10,5210	(236)
OMC	4	From 15,1 to 20	15,0582	3,5636	12,6993	(141)
OMC	5	From 20,1 to 25	20,5015	5,1599	26,6242	(65)
OMC	6	From 25,1 to 35	27,6048	7,41[illegible]9	54,9365	(21)

5.3.16 CBR

This parameter has a definite relationship with requilibrium moisture content. As the CBR increases, so equilibrium moisture content decreases (Table 5.19). This is not a linear relationship and as the CBR decreases below 20 per cent, the equilibrium moisture content increases fast.

5.3.17 Percentage material finer than 0,425 mm

There does not exist a strong correlation between this parameter and equilibrium moisture content (Table 5.20). The general trend is for equilibrium moisture content to increase as percentage finer than 0,425 mm increases, as expected.

TABLE 5.19: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY MAXIMUM CBR

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
MaxCBR	1	From 0 to 5	20,1468	8,1638	66,6473	(88)
MaxCBR	2	From 5,01 to 10	13,7215	4,1216	16,9879	(209)
MaxCBR	3	From 10,01 to 15	10,6449	3,6767	13,5181	(210)
MaxCBR	4	From 15,01 to 20	9,1305	3,7840	14,3190	(141)
MaxCBR	5	From 20,01 to 25	8,5466	3,0414	9,2502	(146)
MaxCBR	6	From 25,01 to 30	7,6324	3,1018	9,6211	(74)
MaxCBR	7	From 30,01 to 35	6,3903	2,6168	6,8477	(72)
MaxCBR	8	From 35,01 to 40	6,5094	2,4701	6,1015	(32)
MaxCBR	9	From 40,01 to 45	7,0455	2,4976	6,2382	(33)
MaxCBR	10	From 45,01 to 50	6,2667	1,5765	2,4853	(21)
MaxCBR	11	From 50,01 to 55	5,7895	2,4882	6,1910	(19)
MaxCBR	12	From 55,01 to 60	5,4167	1,1514	1,3257	(6)
MaxCBR	13	From 60,01 to 65	4,7200	1,0402	1,0820	(5)
MaxCBR	14	From 65,01 to 70	5,6850	1,3466	1,8134	(6)
MaxCBR	15	From 70,01 to 75	7,3000	0	0	(1)
MaxCBR	16	From 75,01 to 80	3,9000	2,2627	5,1200	(2)
MaxCBR	17	From 80,01 to 85	4,5750	3,2827	10,7758	(4)
MaxCBR	18	From 85,01 to 90	7,1000	0	0	(1)
MaxCBR	19	From 90,01 to 95	4,8000	0	0	(1)
MaxCBR	22	From 105,01 to 110	5,0000	0	0	(1)

TABLE 5.20: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY PERCENTAGE MATERIAL FINER THAN 0,425 mm

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
% - 0,425	1	From 0 to 5	10,5100	4,9374	24,3778	(20)
% - 0,425	3	From 10,01 to 15	5,9750	1,0813	1,1692	(4)
% - 0,425	4	From 15,01 to 20	6,5250	2,9757	8,8548	(12)
% - 0,425	5	From 20,01 to 25	6,9250	3,3814	11,4342	(32)
% - 0,425	6	From 25,01 to 30	7,4490	2,7824	7,7416	(41)
% - 0,425	7	From 30,01 to 35	7,7631	4,7327	22,3983	(51)
% - 0,425	8	From 35,01 to 40	8,1444	3,3542	11,2507	(72)
% - 0,425	9	From 40,01 to 45	8,7908	3,5460	12,5743	(65)
% - 0,425	10	From 45,01 to 50	8,6666	3,1589	9,9783	(53)
% - 0,425	11	From 50,01 to 55	9,4794	3,9086	15,2769	(68)
% - 0,425	12	From 55,01 to 60	9,7852	5,1142	26,1552	(54)
% - 0,425	13	From 60,01 to 65	9,5232	4,1020	16,8262	(56)
% - 0,425	14	From 65,01 to 70	11,0937	5,0795	25,8016	(63)
% - 0,425	15	From 70,01 to 75	10,3176	5,1621	26,6472	(85)
% - 0,425	16	From 75,01 to 80	11,1374	5,3207	28,3094	(115)
% - 0,425	17	From 80,01 to 85	11,1816	5,1238	26,2532	(114)
% - 0,425	18	From 85,01 to 90	14,6855	7,0080	49,1127	(76)
% - 0,425	19	From 90,01 to 95	15,7041	7,1049	50,4793	(74)
% - 0,425	20	From 95,01 to 100	17,1059	7,9481	63,1731	(17)

5.3.18 Percentage material finer than 0,075 mm

As may be expected, the equilibrium moisture content increases as the percentage finer than 0,075 mm increases, although a strong correlation does not exist (Table 5.21).

TABLE 5.21: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY PERCENTAGE MATERIAL FINER THAN 0,075 mm

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
% - 0,075	1	From 0 to 5	10,2905	4,9164	24,1709	(21)
% - 0,075	2	From 5,01 to 10	4,6847	1,9964	3,9858	(19)
% - 0,075	3	From 10,01 to 15	6,6355	2,7702	7,6741	(76)
% - 0,075	4	From 15,01 to 20	6,7393	2,5646	6,5772	(110)
% - 0,075	5	From 20,01 to 25	7,9914	3,5640	12,7024	(151)
% - 0,075	6	From 25,01 to 30	8,9909	3,5009	12,2566	(132)
% - 0,075	7	From 30,01 to 35	9,5162	3,3615	11,2996	(105)
% - 0,075	8	From 35,01 to 40	10,1719	3,2727	10,7108	(102)
% - 0,075	9	From 40,01 to 45	11,4203	3,8291	14,6619	(79)
% - 0,075	10	From 45,01 to 50	12,0681	4,5314	20,5337	(69)
% - 0,075	11	From 50,01 to 55	13,9864	5,1972	27,0108	(59)
% - 0,075	12	From 55,01 to 60	15,4923	5,4678	29,8970	(39)
% - 0,075	13	From 60,01 to 65	17,2031	5,5669	30,9906	(32)
% - 0,075	14	From 65,01 to 70	18,3042	6,3707	40,5856	(24)
% - 0,075	15	From 70,01 to 75	20,7720	6,2699	39,3121	(25)
% - 0,075	16	From 75,01 to 80	20,2267	7,7070	59,3978	(15)
% - 0,075	17	From 80,01 to 85	24,1100	8,7822	77,1277	(10)
% - 0,075	18	From 85,01 to 90	13,7667	3,4962	12,2233	(3)
% - 0,075	19	From 90,01 to 95	19,0000	0	0	(1)

5.3.19 Mean annual rainfall

This parameter does not correlate well with equilibrium moisture content, although there is a tendency for equilibrium moisture content to increase as mean annual rainfall increases (Table 5.22).

TABLE 5.22: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY MEAN ANNUAL RAINFALL

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
Mar	1	From 0 to 6 dm	9,3183	5,8990	34,7987	(282)
Mar	2	From 6,1 to 7 dm	10,5763	5,3961	29,1182	(470)
Mar	3	From 7,1 to [illegible] dm	11,2060	5,2048	27,0895	(285)
Mar	4	From 8,1 to 10 dm	13,5632	5,4523	29,7280	(19)
Mar	5	From 10,1 to 12 dm	10,7000	0	0	(1)
Mar	6	From 12,1 to 15 dm	13,2200	4,5639	20,8289	(15)

5.[illegible].20 Climatic N-ind[illegible]

This parameter does not correlate well with equilibrium moisture content, alt[illegible] there is a tendency for equilibrium moisture content to decrease as [illegible] alue increases (Table 5.23). This is not a linear trend, because of the nature of the index.

TABLE 5.13: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY CLIMATIC N-INDEX

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5029	5,5326	30,6093	(1072)
N	1	From 0 to 2	11,1824	4,4966	20,2195	(131)
N	2	From 2,1 to 2,5	11,0940	5,9310	35,1769	(117)
N	3	From 2,6 to 3	10,6853	5,8884	34,6735	(372)
N	4	From 3,1 to 3,5	10,3479	5,1950	26,9880	(138)
N	5	From 3,6 to 4	11,8462	5,9095	34,9228	(80)
N	6	From 4,1 to 5	8,4424	5,0386	25,3877	(132)
N	7	From 5,1 to 6	10,1098	5,0639	25,6431	(102)

5.3.21 *Climatic I-index*

This parameter does not correlate well with equilibrium moisture content, although there is a tendency for equilibrium moisture content to increase as I-value increases (Table 5.24). These trends or lack of trends in the climatic parameters can again be explained in terms of the method of wetting up. In areas where the water table is close to the surface, climate will have only a minor effect on the subgrade moisture regime. Unfortunately the depth to the water table was not recorded and so the influence of climate cannot be studied in more detail.

TABLE 5.24: DESCRIPTION OF EQUILIBRIUM MOISTURE CONTENT BROKEN DOWN BY CLIMATIC I-INDEX

Variable	Code	Value label	Mean	Std. dev.	Variance	N
For entire population			10,5241	5,5525	30,8300	(1057)
I	1	From -45 to -20	9,2579	5,6736	32,1901	(247)
I	2	From -19 to -10	9,9461	5,6824	32,2892	(170)
I	3	From -9 to 0	11,0294	5,7018	32,5111	(477)
I	4	From 1 to 10	10,8908	3,6212	13,1129	(87)
I	5	From 11 to 20	11,6357	4,4783	20,0554	(56)
I	6	From 21 to 40	13,6800	6,8500	46,9220	(5)
I	7	From 41 to 105	14,5267	4,9391	24,3950	(15)

The relationship between the material properties and the equilibrium moisture content may be summarised as in Table 5.25.

TABLE 5.25: SUMMARY OF APPROXIMATE RELATIONSHIPS BETWEEN EQUILIBRIUM MOISTURE CONTENT AND SOIL PARAMETERS

Variable	Correlation coefficient	Relationship	Accuracy
OMC	0,84	EMC < OMC	90 % of the time
MDD	-0,81	$EMC \leq 75 - \frac{1}{30}$ (MDD)	90 % of the time
LL	0,77	EMC ≤ 0,3[illegible] + 1,5 (EMC always < LL)	95 % of the time
LS	0,76	EMC = LS + 5	Usually
PI	0,75	For PI < 10, EMC > PI	Usually
		For PI > 10, EMC < PI	Usually
(%-0,075)	0,68	$EMC < \frac{1}{3}$ (%-0,075) + 5	90 % of the time
PL	0,60	Only about 5 % of local subgrades reach their PL	
CBR	-0,55	If CBR < 20 then EMC > 10	Usually
(%-0,425)	0,41	$EMC < \frac{1}{2}$ (%-0,425)	90 % of the time

5.3.22 General conclusions

The conclusions listed in this paragraph may be divided into three categories:

(a) conclusions drawn from the evaluation of existing models;

(b) conclusions about the influence of non-ratio parameters on subgrade moisture conditions;

(c) conclusions about the influence of ratio parameters on subgrade moisture conditions.

5.3.22.1 Evaluation of existing models

Table 5.1 summarizes the applicability of existing prediction techniques to local conditions and the following conclusions may be drawn:

(a) The most prominent predictor of moisture conditions in the past was plastic limit (PL). The majority of researchers related plastic limit to the equilibrium moisture condition. It is now clear that plastic limit is not a good predictor of equilibrium moisture content except for certain ranges of plastic limit. The fact that the model developed by Livneh and Ishai (1975) correlates so well with equilibrium moisture content, is solely due to the fact that it is applicable to only a very limited range of plastic limit values. If this formula is to be extended to cover the whole range of plastic limit values, the following formula results:

$$EMC = 0{,}83(PL) - 0{,}0009(PL)^2 - 2{,}1$$

This model has a correlation coefficient of 0,61 and a prediction accuracy of 41 per cent at the 85 per cent level. The fact that plastic limit may only be applied successfully as a predictor for certain ranges of equilibrium moisture content, seriously limits its practical applicability.

(b) None of the available models may be applied successfully to local conditions without modifications, because of lack of accuracy. Although the quoted accuracies in Table 5.1 for the existing models may not be compared directly to the accuracies quoted for local conditions because of possible capitalization of chance, the fact that more than 1 100 sample points were considered means that it is unlikely that the pairs of data are incomparable.

(c) Proctor optimum moisture content is the single best predictor of equilibrium moisture content and it correlates significantly better with equilibrium moisture content than any other predictor.

(d) Because none of these models could be used to predict moisture conditions successfully in southern Africa, it was decided to develop more accurate models from a much larger data base including many more parameters.

5.3.22.2 Influence of non-ratio parameters

Because the normal statistical parameters do not describe these variables and their influence on equilibrium moisture content at all, they must be analysed by breaking them down into categories as done in this chapter. The average equilibrium moisture content of different categories may now be studied and compared with other categories and conclusions drawn about the importance or influence of the categories. Alternatively these parameters may be included in the regression analysis, which is strictly theoretically not justified, but which gives acceptable results if consistent coding is done. Thirdly, one can do separate regression analyses on different categories and in this way establish prediction models applicable to different situations. This procedure is theoretically more correct. All three of these methods have been investigated and employed without meeting much success. The reason for this was that a key parameter, depth to the water table, was not measured.

In order to derive more specific trends (which should have been done by correlation with the depth to the water table) it was decided to divide the data into two categories according to climate (because data about the depth to the water table were not available). Too little information about different climatic regions is available for a more elaborate classification. The following criteria were used to define a wet and dry region:

Wet region: MAR $\geq$ 700 mm
$N \leq 2{,}6$
$I \geq -2$

Dry region: MAR < 700
$N > 2{,}6$
$I < -2$

Comparing the data for these two ranges the following conclusions could be drawn:

(a) The elevation of the road surface above the surrounding area does not influence the equilibrium moisture content in the subgrade significantly in any climatic region.

(b) It was found that as the quality of surface drainage improves, so the equilibrium moisture content decreases in a wet climate. It is felt instinctively that this should hold for all climatic regions, but it was found that surface drainage does not play a significant part in arid regions. This may be due to the fact that surface drainage is only one component of a drainage strategy; the subsurface drainage may for example be good while the surface drainage is poor and thus obscure the influence of surface drainage. It is felt that a more detailed investigation of specific drainage situations should be made before conclusive remarks about the influence of drainage on failure, for example, may be made. It must also be borne in mind that the trends described here are very general because they have been derived from general climatic and pavement conditions.

(c) Although the quality of surface drainage does not influence the subgrade equilibrium moisture content significantly in all climatic regions, it has a significant effect on the condition of the road. There is a trend for the road condition to deteriorate as the quality of surface drainage deteriorates.

(d) In general the crack condition of the surfacing is not significantly related to the subgrade equilibrium moisture content, but it was found when considering different climatic regions that the condition or degree of cracking of the pavement does not influence the equilibrium moisture content of the subgrade significantly, although there is a trend for the equilibrium moisture content to be lower in a dry climate if the pavement is cracked. This may be due to the time of year sampling took place. It is also possible in some cases that cracking is only related to the surfacing and that subsequent layers may still be intact, so that as far as the subgrade is concerned, the pavement is still relatively impervious.

(e) In general, the thickness of the bitumen surfacing does not influence the subgrade equilibrium moisture content significantly.

(f) There is a definite relationship between the total thickness of the cover and the subgrade equilibrium moisture content. As the equilibrium moisture content increases, so the cover thickness increases, which can probably be explained by the fact that thicker pavements are required over weaker, wetter soils.

(g) The general condition of the road appears to be a reliable indicator of subgrade moisture accumulation, as such an accumulation usually results in some form of distress. This trend is significant in all climatic regions, but more so in a wet climate. It is also true that the variation in equilibrium moisture content under an unfailed pavement is smaller than that under a failed pavement.

(h) The degree of surface rutting is also related to the subgrade equilibrium moisture content. If a pavement starts rutting, the chances are good that the subgrade is wetting up. This trend was observed in all climatic regions but was more pronounced in the dry areas. This trend seems logical, as rutting is related to subgrade deformation.

(i) There is a trend in the subgrade for the material grading to become coarser as the elevation changes from flat to fill.

(j) There is a clear tendency to provide better quality surface drainage to thicker pavements, which is sensible, because thick pavements usually occur over water-sensitive materials and the benefit of good drainage in this case is both to reduce the subgrade equilibrium moisture content and to prevent the road condition from deteriorating. Good quality surface drainage also prevents thin pavements from deteriorating, but it does not influence the equilibrium moisture content.

(k) It was found that the plastic limit was related to the quality of surface drainage. The trend is for the plastic limit to decrease as the quality of surface drainage deteriorates. This may indicate that where the plastic limit is low, people do not give much attention to surface drainage.

(l) From the analysis it is clear that firstly the condition of rutting and secondly the degree of cracking is related to the condition of the road. It appears that these are the two main external features indicating whether failure is under way or not. Other factors which are related to the incidence of failure are Atterberg Limits, optimum mosture content, maximum dry density and CBR. By keeping the optimum moisture content and Atterberg limits down and by maximizing the maximum dry density and CBR, failure may largely be prevented.

(m) A definite influence of climate on crack formation was observed, in that cracks form more easily and are more abundant in dry climates.

(n) There is a trend for the degree of cracking to increase as the rutting becomes worse.

(o) It was found that the only soil parameter that influenced rutting significantly, was plastic limit.

(p) There is a trend in South Africa to use thicker bitumen surfacings in wet climates, which makes sense, because the analysis showed that thick bitumen surfacings tend to rut and deteriorate more easily in dry climates and tend to last longer in wet climates.

(q) It was found that the climate only influenced the following subgrade soil parameters:

(i) The subgrade equilibrium moisture content tends to increase with incr. ing mean annual rainfall.

(ii) Cracks in the surfacing are less likely to form in wet climates.

(iii) Thicker bitumen surfacings are used in wet climates, so it may be that cracks do happen but do not come through to the surface.

(iv) Grading modulus tends to decrease with increasing mean annual rainfall.

(v) The plastic limit tends to increase with increasing mean annual rainfall.

(vi) The liquid limit tends to increase with increasing mean annual rainfall.

The above are only trends and they are not significant enough to be incorporated in prediction models. It is interesting to note that there is no trend for more failures to occur in a wet climate than in a dry climate.

(r) It is important to note that for a dry climate (mean annual rainfall < 200 mm) there is no significant relationship betwee. equilibrium moisture content and the best predictors for mild climates, with the exception of CBR. One, for example, cannot for an arid climate say (at the 1 per cent level) that as the optimum moisture content increases so the equilibrium moisture content will increase according to a certain formula. For an arid climate there is no significant relation, even at the 5 per cent level, between the Atterberg limits and equilibrium moisture content.

5.3.22.3 General conclusions from analysis about ratio parameters

(a) It was found, when checking the interaction between climate, Atterberg limits and optimum moisture content, that although there was no very strong interaction, there was a trend in the wetter regions for equilibrium moisture content to be less sensitive to Atterberg limit changes, whereas in the dry climates the equilibrium moisture content tends to change fast with changing Atterberg limits and changing optimum moisture content.

(b) There is a general trend for the moisture content to increase with increasing plasticity.

(c) The moisture conditions are not very sensitive to climatic changes except where the rainfall is below 200 mm.

(d) It is interesting to note that there is a trend for pavements to become thicker, the wetter the climate. This seems to indicate that, although not always consciously, designers in the past have to a certain extent taken the effect of environment into account when designing pavements. Some may contend that because the influence of climate on pavement behaviour is not very strong when the general condition is considered, it is unnecessary to make special provision for climate. This is not correct, because the provision made at present is not based on research findings wholely applicable to local conditions. The abovementioned trend may also be due to the fact that there is a trend for subgrade material to become weaker (higher Atterberg limits and finer gradings) with increasing rainfall, thus necessitating thicker pavements. It is thus a material-orientated precaution and not a climatic one. Because the climate not only influences the material properties, but also the moisture conditions, it should be included in a [illegible] way in the design procedure.

(e) It was found from the analysis that [illegible] ascribe failure mainly to saturation, as Table 5.2 indicates. [illegible] holds for all climatic regions. Similarly the occurrence of cracking and rutting cannot primarily be ascribed to subgrade saturation. There is, however, a marked trend for the subgrade to reach saturation if the surface drainage is poor, except in dry regions where saturation was never found to occur.

TABLE 5.26: RELATION BETWEEN PAVEMENT CONDITION AND MEASURED MOISTURE CONDITION

Road condition	Not failed	Borderline	Failed
EMC ≤ OMC	76,7 84,0	10,8 74,2	12,5 65,8
OMC < EMC ≤ SAT.	57,7 12,1	15,3 20,2	27,3 27,3
EMC > SAT.	65,5 3,9	14,6 5,6	22,9 6,8

The top figure in each block is the row percentage and the bottom figure is the column percentage.

(f) As the analysis progressed, it became increasingly clear that it is probably over-conservative to assume subgrade or pavement saturation in most climatic regions in southern Africa. Table 5.27 indicates the situation irrespective of whether the pavement is cracked, failed or rutted, and irrespective of the drainage conditions. It is clear from this table that less than 5 per cent of all subgrades reached saturation in the Transvaal at the time of the Burrow survey (March to September 1973). Because of the small number of samples from the Cape and South West Africa, the results are not as highly significant as the Transvaal ones.

TABLE 5.27: SUBGRADE EQUILIBRIUM MOISTURE CONTENTS (EMC) IN RELATION TO PROCTOR OPTIMUM MOISTURE CONTENTS (OMC) AT ALL SITES

Region	Tvl	Cape	S W A
Samples (no.)	1 102	20	28
EMC > Saturation (%)	4,5	25	0
Saturation ≥ EMC > OMC (%)	15,5	40	0
EMC ≤ OMC (%)	20	65	0
EMC OMC (%)	80	35	100

If only sound pavements with good drainage conditions are selected, Table 5.28 results.

TABLE 5.28: SUBGRADE EQUILIBRIUM MOISTURE CONTENTS IN RELATION TO PROCTOR OPTIMUM MOISTURE CONTENTS AT UNFAILED WELL-DRAINED SITES

Region	Tvl	Cape	S W A
Samples (no.)	167	9	8
EMC > Saturation (%)	6	0	0
Saturation ≥ EMC > OMC (%)	5,5	50	0
EMC ≥ OMC (%)	11,5	50	0
EMC < OMC (%)	88,5	50	100

Although the samples from the Cape [illegible]th West Africa are few, it seems that one can improve t[illegible]re conditions (at least in the Transvaal and Cape) by providing good drainage and proper maintenance. Also, while the percentage of subgrades reaching saturation does not decrease in the Transvaal, the percentage exceeding optimum moisture content is about halved.

CHAPTER 6

DIURNAL AND SEASONAL VARIATIONS IN MOISTURE CONDITIONS IN PAVEMENTS

CHAPTER 6

DIURNAL AND SEASONAL VARIATIONS IN MOISTURE CONDITIONS IN PAVEMENTS

CONTENTS

CHAPTER 6

DIURNAL VARIATION IN MOISTURE CONDITIONS IN PAVEMENTS

6.1 Introduction

In Chapter 5 several general moisture trends in pavements were discussed. Another very important moisture trend in pavements will be discussed in this chapter, namely the daily and seasonal suction variation. A substantial amount of experimental work has been done to determine, in the first instance, the diurnal and seasonal variation in suction in pavement layers and, secondly, the associated variation in pavement response for certain types of pavement. Much work remains to be done before definite recommendations may be forwarded, but such important observations have been made to date that the interim findings warrant a separate chapter. It is the aim of this chapter to summarize the work done thus far and to list the conclusions that may be made at this stage.

6.2 Location of experimental sites

Although more sites were instrumented in the Transvaal, on three of them 24-hour experiments were conducted (as explained in the next paragraph). These sites include

(a) the Computer Centre access road on the CSIR campus

(b) the Old Military Road 300 m north of the Menlyn Road intersection on the south-bound lane.

(c) the HVS site on P6-1 between Bapsfontein and Bronkhorstspruit 42,6 km from Bapsfontein on the southwest-bound lane.

The results from the CSIR campus site especially were so encouraging that it was decided to instrument two sites in regions of climatic extremes. To represent a continuously wet region, a location on the national road N2 from Plettenberg Bay to Humansdorp 53,4 km from Plettenberg Bay was selected and, to represent a dry region, chainage 1665 of road 1/2 between Keetmanshoop and Grunau was selected. Both these latter sites were monitored seasonally.

Three sites in Botswana have also recently been instrumented, on the road between Kanya and Jwaneng near Jwaneng diamond mine, but results from these sites are not available as yet.

6.3 Nature of measurements taken

In order to measure the diurnal variation in suction and pavement response it was necessary to take two hourly relevant readings for a period of at least 26 hours. The following readings had to be taken:

(a) temperature readings in the inner and outer wheel tracks at the surface, at ± centre of base, at ± centre of subbase, at ± centre of selected subgrade and at ± 1000 mm depth (by means of electrical thermo-couples);

(b) suction readings at the same locations as (a), excepting the surface, using psychhrometers in the dew point as well as psychrometric modes;

(c) resistivity measurements at the same locations as (a), excepting the surface, using the NITRR resistivity probes;

(d) redius of curvature readings at five stations, one meter apart longitudinally along both the inner and outer wheel tracks. Triplicate readings were taken at each station and the centre station was as close as possible to the instrument stack. The NITRR radius of curvature meter and method of taking measurements were used;

(e) surface deflection measurements in triplicate at the five stations used for curvature measurements, using the Benkelman beam;

(f) air temperature and relative humidity readings.

6.4 Experimental results

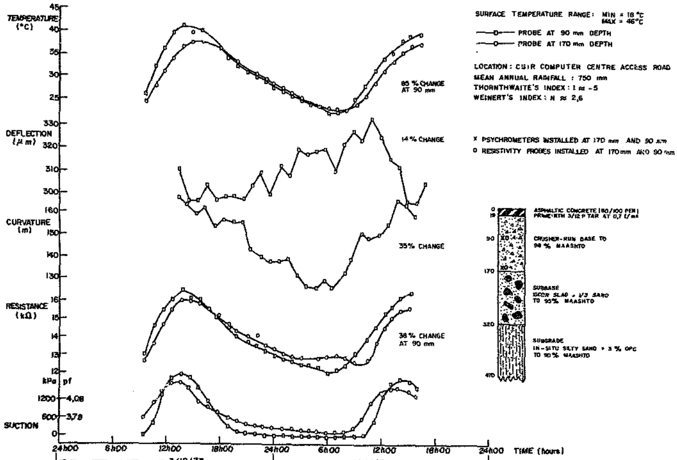


FIGURE 6.1 *COMPARISON BETWEEN TEMPERATURE, SU TION, RADIUS OF CURVATURE AND DEFLECTION IN OUTER WHEELTRACK IN CSIR COMPUTER CENTRE ACCESS ROAD.*

6.4.1 CSIR Computer Centre Access Road

The first site instrumented and monitored was the Computer Centre access road on the CSIR campus. Figure 6.1 is a plot of an experiment carried out on October 3 and 4, 1977 and also shows the profile of the pavement layers. The surface temperature varied between 18 and 46 °C while the sensors at depths 90 and 170 mm had smaller temperature fluctuations. The suction as measured by the psychrometers varied from near zero to over 1 500 kPa. The resistivity probes also showed substantial variation in sympathy with the temperature cycle. The resistivity results are closely related to moisture content, but unfortunately they were not individually calibrated so that no quantitative conclusions could be drawn about actual moisture content changes. The average of the triplicate deflection readings taken at each of the five stations (i.e. average of 15 readings) was plotted hourly and a 14 per cent change ove[illegible] 24 hours period was observed. Similar curvature averages showed [illegible]ent variation over the same period.

This experiment was repeated on two other o[illegible]ions and the results are summarized in Table 6.1. On each occasion similar trends to those shown in Figure 6.1 were produced.

6.4.2 Old Military Road

The second site instrumented was the Old Military Road, approximately 200 m north of the Menlyn Road intersection on the south-bound lane. Only psychrometers were installed at this site in both wheel tracks. The surface temperature was not measured, but the temperature variation at 150 mm depth was 33 per cent. Figure 6.2 shows the results of an experiment carried out on December 20 and 21, 1977, as well as a profile of the pavement layers. The suction at 150 mm depth varied from about 300 kPa, to approximately 3 800 kPa while the radius of curvature varied from 95 m to 165 m, but inversely to the temperature variation. The deflection showed no significant variation. Table 6.2 summarizes the ranges of different variables measured.

TABLE 6.1: SUMMARY OF DAILY RANGES OF SURFACE TEMPERATURE, DEFLECTION, RADIUS OF CURVATURE AND SUCTION AT CSIR COMPUTER CENTRE ACCESS ROAD

Site	Computer Centre Access Road							
Date	3/10/77-4/10/77		2/8/78-3/8/78		10/4/79-11/4/79		10/77 - 4/79	
Wheeltrack	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
Surfacing thickness (mm)	20	20	20	20	20	20	20	20
Surface temperature (°C)		18-46		8-33		15-54		8-54
% change	-	156	-	313	-	248	-	575
Deflection (μm)		295-335		210-240		150-190		1[illegible]0-335
% change	-	14	-	14	-	26	-	3
Radius of curvature (m)		122-165		152-183		204-277		122-277
% change	-	35	-	20	-	36	-	86
Suction in base (90 mm) (kPa)		130-1920	570-1160	150-1725	565-2580	290-2000	565-2580	130-2000
% change	-	1364	103	1050	357	590	357	1439

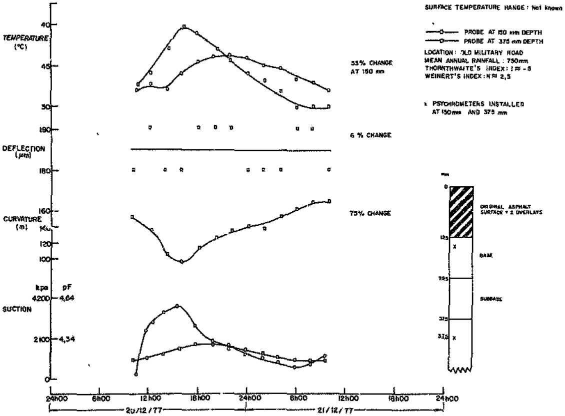


FIGURE 6.2 *COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN OUTER WHEELTRACK IN OLD MILITARY ROAD*

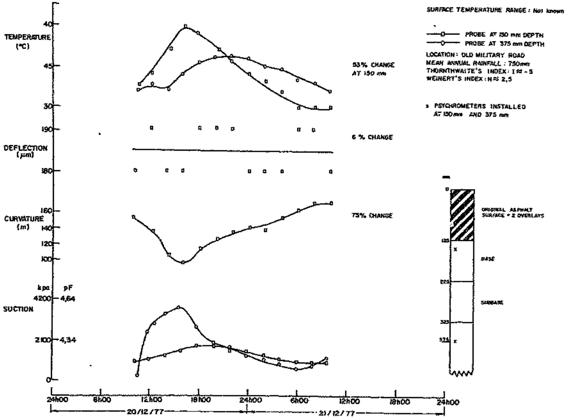


FIGURE 6.2 *COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN OUTER WHEELTRACK IN OLD MILITARY ROAD*

TABLE 6.2: SUMMARY OF DAILY RANGES OF SURFACE TEMPERATURE, DEFLECTION, RADIUS OF CURVATURE AND SUCTION AT OLD MILITARY ROAD

Site	Old Military Road
Date	20-12-77/21-12-77
Wheeltrack	Outer
Surfacing thickness (mm)	125
Bitumen temperature °C at 150 mm % change	30-46 53
Deflection (mm) % change	180-190 6
Radius of curvature % change	95-165 74
Suction in base kPa (150 mm) % change	300-3825 1175
Suction in subgrade kPa (375 mm) % change	1000-1800 60

6.4.3 P6-1 near Bronkhorstspruit

The third site instrumented in the Pretoria region was road P6-1 near Bronkhorstspruit. This site was very wet during the experiment on December 11 and 12, 1979. The water table was only 2,2 m from the surface and positive as well as negative water pressures were recorded, using tensiometers, as the psychrometers could not detect such low suctions. Figure 6.3 shows the results of the experiment as well as a profile to a depth of 2 m. The psychrometers and resistivity probes indicated little daily variation and continuously wet conditions. The curvature and deflection results show that pavement response did not change much over the 24-hours period. Table 6.3 summarizes the ranges of the various variables monitored.

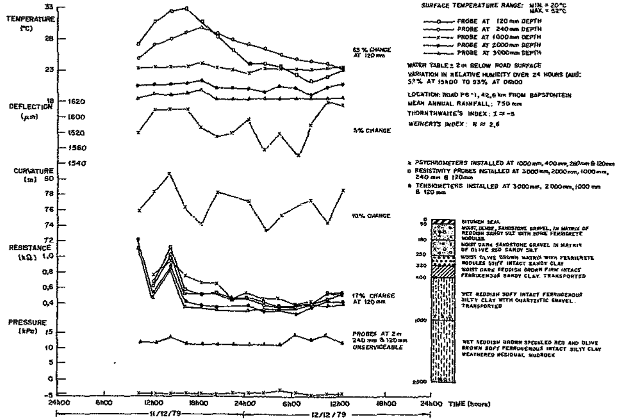


FIGURE 6.3 *COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN INNER WHEELTRACK OF HVS TEST SITE P6-1 NEAR BRONKHORSTSPRUIT*

TABLE 6.3: SUMMARY OF DAILY RANGES OF SURFACE TEMPERATURE, DEFLECTION, RADIUS OF CURVATURE AND SUCTION AT P6-1 NEAR BRONKHORSTSPRUIT

Date	11-12-79/12-12-79	
Wheeltrack	Inner	Outer
Surfacing thickness (mm)	50	50
Surface temperature °C	-	20-52
% change		160
Deflection (µm)	155-165	185-195
% change	6	5
Radius of curvature	54-60	66-81
% change	11	23

6.4.4 Storms River

As stated before, it was decided to instrument sites which represent regions of climatic extremes. To represent a continuously wet area, a site on the road N2 near Storms River was chosen. Both wheel tracks at this site were fully instrumented with psychrometers, resistivity probes and thermocouples. The site has been visited four times to conduct a full 24 hours experiment, once in each season. Figure 6.4 shows the results of a typical test in the outher wheel track, while Figure 6.5 shows a typical result for the inner wheel track. The figures also show a profile of the local pavement layers as well as a description of the climate. Table 6.4 summarizes the ranges of variables measured at this site.

6.4.5 Keetmanshoop

Keetmanshoop was selected to represent a dry area. This site was instrumented initially with psychrometers, resistivity probes and

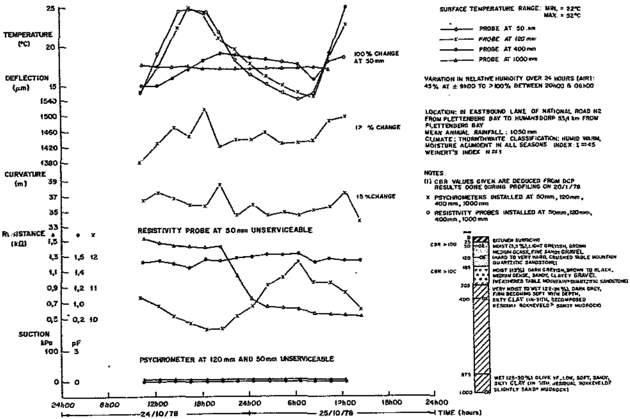


FIGURE 6.4

COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN OUTER WHEELTRACK NEAR STORMSRIVER

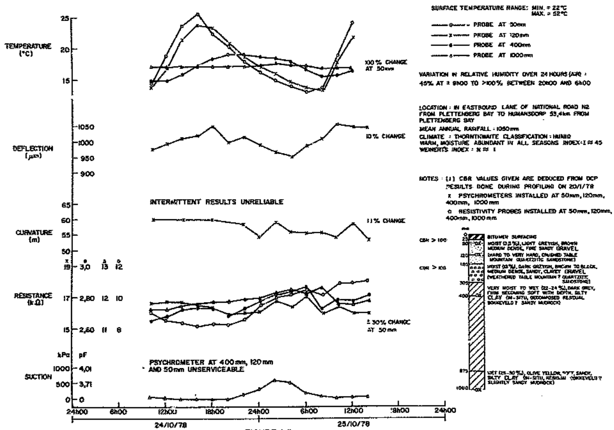


FIGURE 6.5

COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN INNER WHEELTRACK NEAR STORMSRIVER

TABLE 6.4: SUMMARY OF DAILY RANGES OF SURFACE TEMPERATURE, DEFLECTION, RADIUS OF CURVATURE AND SUCTION AT STORMS RIVER

Site					Storms River					
Date	18/7/78-19/7/78		24/10/78-25/10/78		24/1/79-25/1/79		13/6/79-14/6/79		1978-1979	
Wheeltrack	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
Surfacing thickness (mm)	25	25	25	25	25	25	25	25	25	25
Surface temperature (°C)		5,8-25,3		10,8-41,3		22-62		6-22,6		5,8-62
% change	-	336	-	282	-	182	-	277	-	970
Deflection (µm)	940-990	1240-1300	960-1060	1380-1540	1100-1160	1450-1520	900-970	1320-1410	900-1160	1240-1540
% change	5	5	10	12	5	5	8	7	29	24
Radius of curvature (m)	54-60	40-44	54-60	33-38	56-60	37-41	54-60	35-39	54-60	33-44
% change	11	10	11	15	7	11	11	11	11	33
Suction in sub-base (90 mm) (kPa)	-	-	-	-	-	-	-	-	-	-
% change	-	-	-	-	-	-	-	-	-	-
Suction in subgrade (1 000 mm) (kPa)	0-0	0-0	-	0-600	270-590	230-680	0-0	0-0	0-590	0-680
% change	0	0	-	-	120	190	-	-	-	-

thermocouples. The site was subsequently visited four times in order to monitor the influence of season on the suction regime. Both the inner and outer wheel tracks were instrumented comprehensively. During each visit measurements were taken as described in paragraph 6.3. Figure 6.6 shows a typical result for the outer wheel track, while Figure 6.7 shows a typical result for the inner wheel track. The figures also show a profile of the local pavement layers as well as a description of the climate. Table 6.5 summarizes the ranges of variables measured at this site.

6.5 Conclusions

6.5.1 CSIR Computer Centre Access Road

As it is clear from Figure 6.1,there is a substantial daily temperature-induced variation in suction, especially in the top pavement layers. This is accompanied by a similar change in pavement response. The minimum curvature roughly coincides with the maximum deflection at around 06h00. It is apparent that the deflection curve is slightly displaced relative to the temperature and curvature curves. This is because deflection is a measure of the deeper pavement layer's performance while curvature is more related to the top pavement layer's performance. If a temperature curve for a depth of ± 400 mm had been drawn, it would relate better with the deflection results, as it would also be displaced because of the time taken for the heat front to influence that depth.

From Table 6.1 it can be seen that the deflection during the first experiment was about 0,3 mm while it was about half that during the last visit. A similar trend is present in the radius of curvature. These significant increases in pavement strength cannot be explained in terms of the environment. It is probably due to a strengthening of the stabilized sub-base and subgrade with time.

It is a well-known fact that asphaltic pavements become softer the higher their temperatures. The direct opposite trend is seen here where the maximum temperature in a layer causes the maximum suction, which results in a maximum curvature and a minimum deflection. This specific pavement

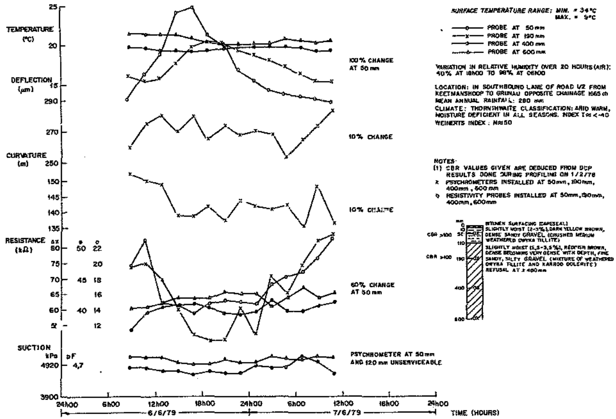


FIGURE 6.6 *COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN OUTER WHEELTRACK AT KEETMANSHOOP 50,2 km SOUTH OF KEETMANSHOOP*

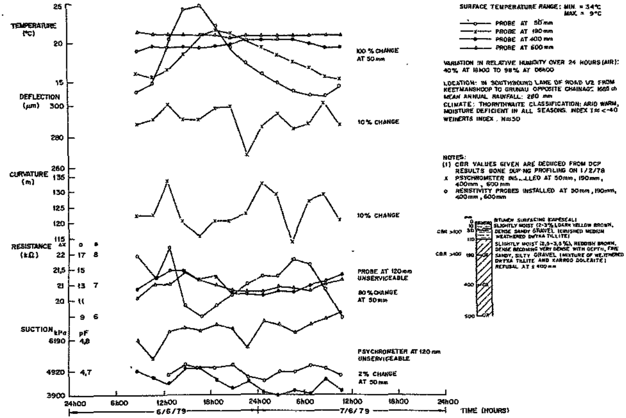


FIGURE 6.7 *COMPARISON BETWEEN TEMPERATURE, SUCTION, RADIUS OF CURVATURE AND DEFLECTION IN INNER WHEELTRACK AT KEETMANSHOOP 50,2 km SOUTH OF KEETMANSHOOP*

TABLE 6.5: SUMMARY OF DAILY RANGES OF SURFACE TEMPERATURE, DEFLECTION, RADIUS OF CURVATURE AND SUCTION AT KEETMANSHOOP

Site	Keetmanshoop									
Date	11/7/78-12/7/78		17/10/78-18/10/78		31/1/79-1/2/79		6/6/79-7/6/79		1978-1979	
Wheeltrack	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer	Inner	Outer
Surfacing thickness (mm)	10	10	10	10	10	10	10	10	10	10
Surface temperature (°C)		8-32		15,5-57		28-64,5		9-34 5		8-64,5
% change	-	300	-	267	-	130	-	283	-	706
Deflection (μm)			300-330	300-340	280-310	280-310	270-310	260-290	270-330	260-340
% change	-	-	10	13	11	11	15	12	22	30
Radius of curvature (m)			108-127	113-130	118-132	124-140	115-132	135-153	108-132	113-153
% change	-	-	18	15	12	13	17	13	22	35
Suction of sub-base (90 mm) (kPa)		1858-2464	-	2552-3564	-	3644-5267	-	-	-	1858-5267
% change	-	32	-	39	-	44	-	-	-	183
Suction in subgrade (600 mm) (kPa)	5182-6064	4278-4382	5764-6473	4470-4628	6188-6479	4480-4694	5390-7439	5030-5775	5182-6479	4287-5775
% change	17	2	12	4	5	5	38	14	25	35

thus gets stiffer with increasing temperature. It is expected that the trend will be even more pronounced if the opposite influence of the thin asphaltic concrete layer could be eliminated.

This observation is obviously highly significant and merits more investigation, as it may have extremely important practical implications. For instance, the life of a road is approximately inversely proportional to the fourth power of the deflection and approximately proportional to the fourth power of the curvature, and this observation is commonly used as a criterion for pavement evaluation with the view to possible maintenance. This means serious errors in the prediction of pavement life may occur if cognizance of the diurnal performance variation is not taken. Another potential practical benefit includes the prohibition of heavy traffic on sensitive roads during certain hours of the day. It is clear that these pavements are capable of carrying much more traffic, provided the bulk of this traffic is experienced during the "strong" hours of the pavement.

6.5.2 Old Military Road

Seeing that such important observations have been made at the Computer Centre access road, the Old Military Road, which has a totally different construction, was instrumented to bear out the initial observations. From inspection of Figure 6.2 there appears to be some serious discrepancies. The surface deflection, for instance, does not change significantly as the suction and temperature changes.

As surface deflection is a measure of the vertical strain in the subgrade, it is used as a measure to evaluate the deeper pavement layers as explained before. In these layers the change in suction, although significant, is not very great and it is thus not entirely surprising that no daily change in deflection could be detected. It is also possible, for this type of pavement, that the total surface deflection measured at the surface is made up of deformations of more than one layer. It may not be solely attributable to the subgrade deformation. The bituminous surfacing is known to soften with increased temperature while the subgrade will strengthen with increased temperature, and total deflection of the surface may thus stay constant. In order to explain this phenomenon more accurately, a structural analysis of the pavement should be done using elastic parameters derived from DCP tests and comparing the calculated deflections with the measured deflections. In this way one should be able to assign a percentage of the total deflection to each layer and relate that to suction changes in that layer. It is worthwhile to point out here that the installation of multi-depth deflectometers at psychrometer sites will be of great assistance in interpreting the pavement response results.

The radius of curvature, which is an indication of the radial strain at the bottom of the surfacing, is used to evaluate the top pavement layers where the temperature and suction changes are more significant. One may therefore expect that this parameter should show a better relationship with suction changes, as these are more pronounced in the top layers. From Figure 6.2 it is clear that there is a large variation in radius of curvature (twice the variation on the CSIR campus) but that it follows a trend opposite to that of suction. The variation in radius of curvature can thus not be explained in terms of the suction variation. This variation is caused by the variation in elasticity properties of bitumen with temperature. The influence of this thick temperature-sensitive bitumen surfacing completely obscures the influence of suction changes on the elastic properties of the base. The temperature variation at a depth of 150 mm is very similar to that obeserved at 170 mm depth before, with peaks occurring slightly later because of the season change. Another similarity is the suction variation in the base at a depth of 150 mm, which corresponds with the variation detected at 170 mm before. This may be regarded as confirmation that large suction variations do occur in pavement layers on a daily basis, but that the influence of this variation on pavement response depends on the type of pavement under consideration.

Another interesting observation that may be made from Figure 6.2 is that there is a marked reduction in temperature and suction variations at depths exceeding 400 mm. Other data also indicated that the temperature in the centre of the pavement is fractionally higher than that in the outer wheel track. It is also true that the outer wheel track heats up more slowly and to a lesser extent that the inner wheel track, although both positions cool down to about the same level.

It is also observed that the variation in suction in the centre of the pavement is larger than that in the outer wheel track, which ties in with the observation that the temperature varies more in the centre line than in the outer wheel track. The magnitude of the variation is also of extreme significance, because the moisture condition in the base in the centre of the road varies from near saturation to a fairly dry state in 24 hours. This clearly indicates that for this type of pavement the influence of temperature on suction in the base is so important that no equilibrium conditions can be achieved in the base. It also indicates that *for this type of pavement the base approaches* saturation in the early morning hours. It should, however, be noted that more stable conditions are achieved at deeper levels.

At deeper levels in the pavement the magnitude of the suction variation is again larger in the centre than in the outer wheel track, but the general level of suction is lower, which means that the pavement is generally wetter in the centre. This does not tie in with the temperature variations, because the centre of the pavement is at a higher temperature and should therefore be at a higher suction. The phenomenon may, however, be explained by the classic wetting up mechanism of materials under covered areas.

6.5.3 P6-1 near Bronkhorstspruit

Only one twenty four hour experiment was conducted at this site, in December 1979, which happened to be an above normal wet summer and the water table rose to within about two meters of the road surface. These wet conditions rendered the psychrometers inoperative, as they cannot measure suctions below $\pm$ 50 kPa accurately. The water pressure readings plotted on Figure 6.3 were thus measured with the use of tensiometers. It is interesting in this case to note the accuracy and reliability of the tensiometers, as they monitored a positive pressure of about 13 kPa at a depth of 3 m, which corresponds to a depth of about 1,3 m beneath the water table, where it in fact was. A negative pressure (i.e. suction) of about 5 kPa was monitored at a depth of 1 m, which corresponds to a height of about 0,5 m above the water table, which again was in fact the case.

What is clear from this exercise is that under such conditions of high degrees of saturation, a significant temperature change will not produce associated suction changes. This means that if a pavement layer has a high degree of saturation, its strength will not fluctuate significantly on a daily basis, as can be seen from the curvature and deflection variations in Figure 6.3.

Although this pavement construction is again different i.om the two reported before and therefore not directly comparable, it is clear that in this wet state the deflections exceed 1,5 mm and the curvatures are exceedingly small, which indicates that the pavement is relatively weak and close to failure.

6.5.4 Storms River

As may be seen from Figures 6.4 and 6.5 the main feature is that the suction does not change much; in fact, it remains below about 50 kPa in the outer wheel track. It should be pointed out that it is not correct to assume that the suction remains constant if the psychrometers continually read zero, because they are insensitive to the low suction ranges. One may only deduce that any variation, if it takes place, never exceeds a peak of about 50 kPa. The rise in suction during the night in the inner wheel track may not be explained in terms of a temperature change, as there wa. virtually no temperature variation at that depth. The fact that it was the only psychrometer operative in the inner wheel track means that it could not be checked against trends monitored by other instruments.

What is, however, confirmed by every experiment carried out near Storms River is that in a continuously wet pavement, large surface temperature variations (more that 100 per cent) will not result in large suction or moisture changes in any pavement layer. It is also not accompanied by large variations in pavement response. The seasonal variations in curvature in the inner wheel track is of the same order as the daily variation, while that variation in the outer wheel track is about double the daily variation. The daily curvature variation is between 10 and 15 per cent, while the seasonal variation may be as high as 30 per cent, with the lowest deflections and highest curvatures during the cold months. Daily deflection variations between 5 and 12 per cent were measured while the seasonal variations ranged between 25 and 30 per cent, which is more than double the daily variation. It is also significant that the deflection in the outer wheel track is about 30 per cent higher than that in the inner wheel track throughout the year, probably because of water ingress through the shoulders and side drains.

6.5.5 Keetmanshoop

From inspection of Figures 6.6 and 6.7 and Table 6.5 it is clear that although quite large variations in suction, specifically in the sub-base occur, all pavement layers generally remain fairly dry throughout the year. These variations are also random and not attributable to temperature changes. It is also clear that for this type of pavement in this very dry climate no temperature-related daily variation in pavement response is detected. This means that if the climate is continuously dry, large temperature variations will not cause large suction or pavement response variations in this type of pavement. The seasonal variation in radius of curvature in the inner wheel track is of the same magnitude as the daily variation (12 - 20 per cent), while the seasonal variation in the outer wheel track is about twice the daily variation. Seasonal deflection variations in both wheel tracks are about twice the daily deflection variation. It is also interesting to note that the daily deflection and curvature variations in Keetmanshoop are somewhat larger than the corresponding variations in a wet area like Storms River.

6.5.6 Summary of conclusions

Although specific conclusions have been drawn when considering each experimental site, it will serve a useful purpose to summarize them here.

(a) It has been shown in four different experiemnts that substantial temperature-induced pavement response variations occur on a daily basis. These variations may be directly related to suction variations. The maximum suction is reached when the temperature in that layer reaches a maximum, usually around 14h00. The load-carrying capacity of the pavement is highest during this time. A minimum suction is reached at around 06h00, when the upper ± 400 mm of pavement reaches a minimum temperature. Heavy loads applied during this time cause most damage.

(b) The top pavement layers (± 400 mm) are subjected to large temperature fluctuations and therefore larger suction changes. This explains why the curvature variation is larger than the deflection variation, as the curvature is a measure of the radial strain at the bottom of the surfacing while the deflection is a measure of the vertical strain in the subgrade.

(c) Daily temperature changes of more than 50 per cent in certain pavement layers are sufficient to cause substantial suction changes.

(d) The softening of a thin bitumen surfacing with increasing temperature is more than compensated for by the stiffening of a granular base with increasing temperature in cases where the surfacing is about 20 mm thick. When the asphaltic layer is more than 100 mm thick, this is no longer true.

(e) The influence of different pavement constructions is not known at this stage, as individual pavement layers and their specific contribution to total surface deflection have not been studied. It is, however, known that an unsaturated granular base, subjected to a 50 per cent daily temperature variation, will undergo substantial suction changes, which will cause substantial variations in pavement stiffness. This is so because the suction acts as a confining pressure and increases the load-carrying capacity of the base by several orders of magnitude daily.

(f) The term "equilibrium moisture content" may not be applied to the top pavement layers, as large daily changes happen. It is more applicable to subgrade moisture conditions.

(g) This daily pavement response variation is not apparent in continuously wet pavements. No fixed criteria have been developed so far, but where the water table is close to the surface and the mean annual rainfall exceeds 1 000 mm, this variation has not been detected.

(h) This daily pavement response variation is also not apparent in continuously dry pavements. Although no specific criteria have been developed so far, this variation could not be detected in the Keetmanshoop area where the rainfall was less than 300 mm. This may be due to the extremely hard subgrade at the experimental site, as large response variations have been found in other areas in Namibia.

(i) It was found that the daily curvature variation was about half the seasonal variation, while the seasonal deflection variation may be substantially more than the daily variation. This is related to the daily and seasonal temperature variations.

(j) It is recommended that because of the possible practical application of these observations more work should be done in order to quantify some trends. For example, actual moisture content changes should be measured daily using non-destructive methods. A laboratory study should be undertaken to determine the influece of varying temperature and temperature gradients on psychrometer response. The observation that the psychrometer read-out fluctuates as a load passes over it, should be investigated. Multi-depth deflectometers should be installed to quantify the influence of various pavement layers more accurately. The pavement should be analysed using some non-linear elastic analysis method, so that the observed variations in pavement response may be predicted theoretically from changing elastic strength parameters. This should be done in conjunction with a laboratory study to determine the influence of temperature on suction and thus the strength parameters of certain pavement materials.

CHAPTER 7

DEVELOPMENT OF EMPIRICAL PREDICTION TECHNIQUES FOR SOUTHERN AFRICA

CHAPTER 7

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CHAPTER 7

THE DEVELOPMENT OF EMPIRICAL PREDICTION TECHNIQUES FOR SOUTHERN AFRICA

7.1 Introduction

The main aim of this chapter is to report on the development of empirical moisture prediction techniques for southern Africa. It was concluded in Chapter 5 that no existing model is ideally suited to local conditions and because such a wealth of information is available locally, it was fairly easy to develop unique prediction models for local conditions. Various techniques were employed in order to optimize the prediction models and this chapter has as its first aim to report on this development.

The secondary aim is to report on an investigation into the prediction possibilities of other soil parameters that have been carried out for completeness' sake. Preliminary analyses were carried out to establish models for the prediction of the California Bearing Ratio, the compaction characteristics and the Atterberg limits.

7.2 Statistical background

The computer program used in these analyses was the latest version of "Statistical Package for the Social Sciences" (SPSS) which is ideally suited for this kind of analysis and which includes such sophisticated facilities as non-kinear regression options.

An important fact that should be highlighted at this stage is that many of the predictors used in this analysis are not accurately reproduceable and thus include implicit errors of measurement. This means that the classical regression theory may not be applied strictly to these predictors, because it is based on the assumption that predictors are known fairly accurately and with a normal error distribution. Although this assumption does not hold true for some predictors used in this analysis, there is at present no readily available alternative statistical theory to treat uncertainties in both the predictor and the predicted. Experience has, however, shown that if the normal regression theory is used in this case, the resultant error is not unacceptably large (De Jager, 1978). This approach has been used in similar situations in the past with much success in other disciplines, for example psychology. It is therefore considered reasonable to use it here.

Normally the first task of data analysis is to determine the basic distributional characteristics of each of the variables to be used in the subsequent statistical analysis. Information of the distribution, variability and central tendencies of the variables provides the researcher with necessary information required for selection of subsequent statistical techniques. For this reason descriptive statistics were firstly calculated, as reported in Chapter 5.

The next step is to do a multiple stepwise linear regression, which requires that variables be measured on the interval or ratio level and that the relationships among the variables are linear and additive. These restrictions are not absolute, however, and nominal variables can be incorporated into the regression through the use of various techniques (Markham, 1978). They have been incorporated in this analysis and although they do not strictly satisfy the assumptions made about regression, the technique has been successfully applied elsewhere (De Jager, 1978). The nature of the data, on the other hand, made it difficult to carry out a statistical test for linearity, but the plots reproduced in Appendix C clearly show that not all parameters have linear relationships with the moisture content. Various different functions were used to transform the variables and to optimise the transformations. The following transformations were eventually included in the analysis:

Grading modulus transformed to $(\text{Grading Modulus})^{-0,5}$
Sand equivalent transformed to $(\text{Sand Equivalent})^{-0,7}$
Maximum CBR transformed to $(\text{Maximum CBR})^{-0,6}$
Climatic N-value transformed to $(\text{N-value})^{-1}$

All other parameters had linear relationships with moisture content and did not have to be transformed.

7.3 The prediction of subgrade moisture conditions

7.3.1 General linear regression

For easy interpretation of information presented in the following equations, the reader is referred to abbreviations summarized in Appendix A.

As a first step it was decided to do a multivariate stepwise linear regression on all the data. Appendix D shows those results of the analysis of 914 boreholes. The correlation coefficients between all variables are given in tabular form and from this it is immediately apparent that the parameters that correlate highly with equilibrium moisture content also correlate highly with each other. This implies that if one of these is used in a regression equation, little improvement in accuracy will result from adding another, since they explain the same variance.

Table 7.1 clearly indicates this trend of high inter-parameter correlation coefficients.

TABLE 7.1: INTER-PARAMETER CORRELATION COEFFICIENTS

	EMC	OMC	MDD	LL	LS
OMC	0,84				
MDD	-0,82	-0,92			
LL	0,77	0,88	-0,78		
LS	0,76	0,95	-0,76	0,95	
PI	0,76	0,95	-0,76	0,95	0,98

This shows clearly that, although five parameters correlate highly ($r \geq 0,8$) with equilibrium moisture content, little improvement will result from combining the variables because of the very high interparameter correlations. This is obviously a serious drawback. As stated above, these analyses were done in a stepwise manner and the parameters were inserted into the equation in order of importance. In the summary Table D5 in Appendix D, the order in which the parameters appear in the prediction equation is shown. From Table D5 it is clear that any refinement beyond step 2 for model A is of academic significance only and does not materially improve the prediction accuracy. Therefore the full regression details for the first two steps only are given. The practical equation to be used in this case is:

$$EMC = 0{,}79(OMC) + 15{,}92(CBR)^{-0{,}6} - 3{,}0 \quad \ldots\ldots\ldots\ldots \quad (7.1)$$

This model has a multiple correlation coefficient of 0,86 and a standard error of estimate of 2,89, yielding an error at the 85 per cent level of only 26 per cent. Or alternatively, if CBR is not known:

$$EMC = 1{,}1(OMC) - 3{,}4 \quad \ldots\ldots\ldots\ldots \quad (7.2)$$

This model has a multiple correlation coefficient of 0,84 and a standard error of estimate of 3,08, making the error at the 85 per cent level 28 per cent. A regression analysis using only the ratio and interval parameters was also carried out for completeness' sake. Because the first two variables in the analysis are of this type in any case, differences only result in later steps, as shown in Appendix D for model B.

7.3.2 Adjusted Atterberg limits

From past experience and intuition it was felt that there ought to be some relationship between the Atterberg limits and the percentage material finer than 0,425 mm, as these tests are only done on this size fraction. To determine whether this in fact is true, it is necessary to determine whether there is an interaction between liquid limit and the percentage material finer than 0,425 mm, i.e. to determine whether the relationghip between equilibrium moisture content and liquid limit stays constant as the percentage material finer than 0,425 mm changes. Figure 7.1 shows that there is in fact an interaction, wich means that one should combine liquid limit with the percentage material finer than 0,425 mm in some manner to create an extra parameter which will improve the accuracy of prediction.

In the process of developing the optimum combination between the percentage material finer than 0,425 mm and liquid limit, different models were tested and finally the following model emerged:

$$LLT = (LL)^{-0{,}718}(\% - 0{,}425)^{0{,}3089} \quad \ldots\ldots\ldots\ldots \quad (7.3)$$

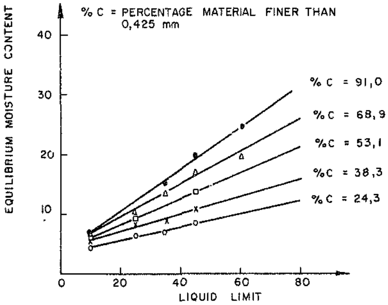

FIGURE 7.1

where LLT = adjusted liquid limit

LT = liquid limit

(% - 0,425) = percentage material finer than 0,425 mm.

To ensure that this model is better than a simple combination of the two parameters, their individual correlation coefficients with moisture content were tested in a t-test of significance, which showed that the adjustment proposed in equation 7.3 is significantly better than a simple combination on the 1 per cent level.

Other soil parameters like linear shrinkage and plasticity index were also treated in ths way. Figures 7.2 and 7.3 indicate that interaction takes place to a lesser extent for both plasticity index and linear shrinkage and it was thus decided to combine them with the percentage material finer than 0,425 mm.

The best adjustments for these variables were found to be:

$$PIT = PI(\% - 0{,}425)^{0{,}5} \qquad (7.4)$$

$$LST = (LS)(\% - 0{,}425)^{0{,}7} \qquad (7.5)$$

where PIT = adjusted plasticity index

LST = adjusted linear shrinkage

and the other vaiables have their normal meanings.

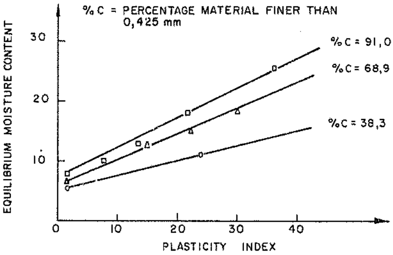

FIGURE 7.2

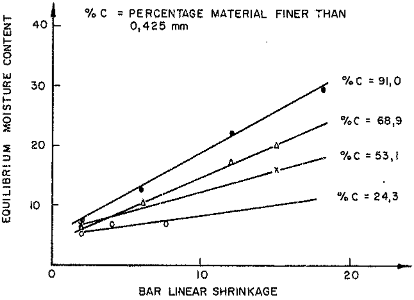

FIGURE 7.3

where PIT = adjusted plasticity index

LST = adjusted linear shrinkage

and the other vaiables have their normal meanings.

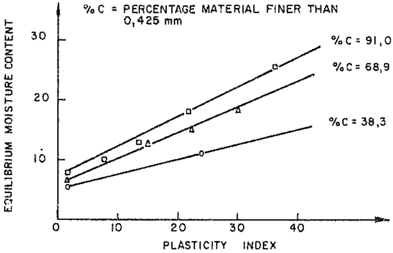

FIGURE 7.2

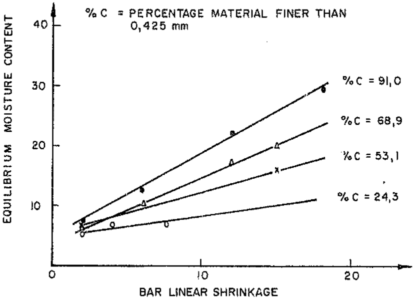

FIGURE 7.3

Because it was not one of the main aims of this study to develop optimum adjustments for parameters other than the equilibrium moisture content, more work should be done to prove that the adjustments given are in fact the best for other soil properties.

It was also intuitively felt that the Atterberg limits might change with climatic conditions, but this was found not to be the case. Figures 7.4 to 7.7 show that there is essentially no interaction between the Atterberg limits and the climatic indices.

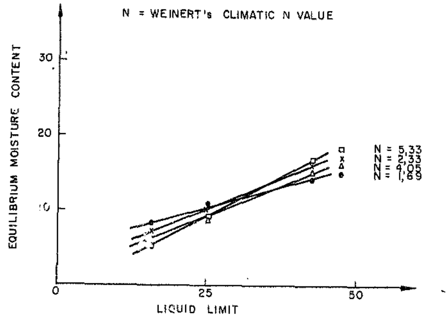

FIGURE 7.4

Various other parameters were tested for interaction but none were detected. An attempt was made to isolate a single optimum adjustment which could be used for all Atterberg limits and linear shrinkages to increase correlations with all other parameters. This could not be done entirely successfully because different adjustments relate best to different soil properties, as can be seen in Table 7.2 It can, however, be seen that any adjustment results in a significant increase in correlation with any specific parameter. Analysing the table shows that the best general adjustment for Atterberg limits and linear shrinkage is:

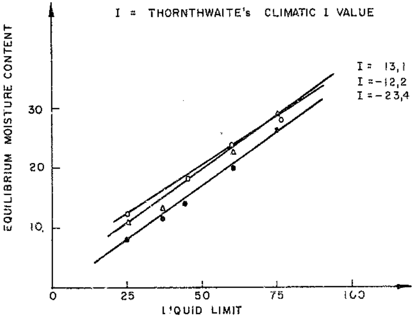

FIGURE 7.5

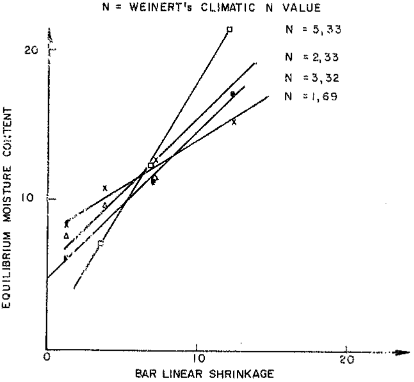

FIGURE 7.6

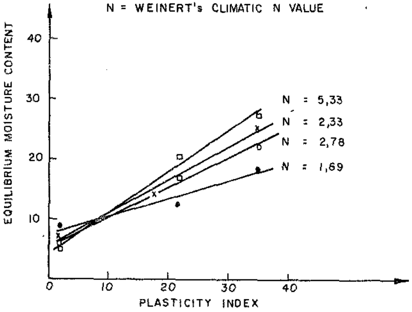

FIGURE 7.7

$$AAL = (AL)^{0,7}(\% - 0,425)^{0,3} \quad \ldots\ldots\ldots \quad (7.6)$$

where AAL = adjusted Atterberg limits or linear shrinkage
AL = Atterberg limit or linear shrinkage
(% - 0,425) = percentage finer than 0,425 mm.

Using the adjustment developed, the best prediction equation for equilibrium moisture content is:

$$EMC = 0,67(OMC) + 0,18(LL)^{0,7}(\% - 0,425)^{0,3} - 4,2 \quad \ldots\ldots\ldots \quad (7.7)$$

This model has a correlation coefficient of 0,85 and a standard error of estimate of 2,98, which means the error at the 85 per cent level is 27 per cent.

7.3.3 Linear regression between certain limits

From Table 7.1 it is clear that optimum moisture content correlates better than any other parameter with equilibrium moisture content. The

TABLE 7.2: CORRELATION COEFFICIENTS OF ADJUSTED ATTERBERG LIMITS WITH VARIOUS PARAMETERS

	Adjustments	EMC	MDD	OMC	CBR
	LL	,771	-,782	,873	-,469
(1)	$LL^{0,7}(\% - 0,425)^{0,3}$	,831	-,904	,920	-,639
(2)	$LL(\% - 0,425)^{0,5}$	,830	-,897	,915	-,608
(3)	$LL(\% - 0,425)$	,806	-,898	,881	-,628
	PI	,756	-,758	,808	-,523
(1)	$PI^{0,7}(\% - 0,425)^{0,3}$	,787	-,833	,841	-,631
(2)	$PI(\% - 0,425)^{0,5}$	,792	-,828	,841	-,581
(3)	$PI(\% - 0,425)$	,786	-,839	,834	-,587
	LS	,757	-,755	,823	-,576
(1)	$LS^{0,7}(\% - 0,425)^{0,3}$	,781	-,816	,846	-,667
(2)	$LS(\% - 0,425)^{0,7}$	,803	-,845	,863	-,619
(3)	$LS(\% - 0,425)^{0,5}$	,795	-,852	,852	-,607
(4)	$LS(\% - 0,425)$	,800	-,852	,857	-,613

next logical step is to determine whether it is in fact true over the whole range of optimum moisture content values. Figure 7.8 shows that at a value of just over OMC = 12, the adjusted liquid limit becomes the most important predictor and therefore it was decided to develop two different sets of equations.

(a) for the optimum moisture content values less than 13; and

(b) for optimum moisture content values greater than or equal to 13.

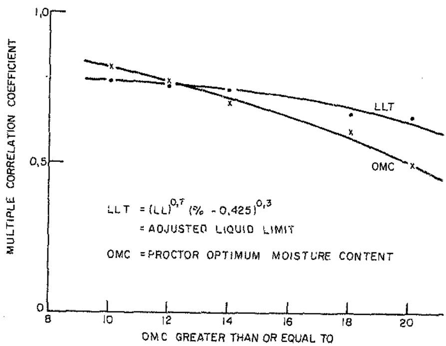

FIGURE 7.8
COMPARISON OF CORRELATION COEFFICIENTS

(a) Linear regression on optimum moisture content values less than 13

A full linear regression analysis was done on all the data values where the optimum moisture content was less than 13. As was expected, optimum moisture content correlated best with equilibrium moisture content, but the second best predictor in this case was sand equivalent, while I-value was the third best predictor. Little accuracy increase resulted from including more parameters, and the best empirical formula is as follows:

$$EMC = 0,78(OMC) - 0,156(SE) + 0,054(I) + 2,84 \quad \ldots\ldots\ldots \quad (7.8)$$

This equation has a multiple correlation coefficient of 0,63 and a standard error of estimate of 2,24. The error at the 85 per cent level is 28 per cent. Although the correlation coefficient is less than 0,8, one must remember that only a fraction of the data is considered and therefore the multiple correlation coefficient calculated from this is not directly comparable with the correlation coefficient calculated for the whole range.

For an indication of the accuracy obtained, one should rather look at the error at the 85 per cent level. Other useful formulae that may be used if all the best predictors are not available, include the following:

$$EMC = 0,87(OMC) - 0,17(SE) - 1,7 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7.9)$$

where the multiple correlation coefficient is 0,59, the standard error of estimate is 2,34 and the error at the 85 per cent level is 29 per cent. Alternatively, because the sand equivalent test is not a common test in the Transvaal:

$$EMC = 0,86(OMC) + 0,07(\% - 0,075) - 3 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7.10)$$

where the multiple correlation coefficient is 0,55, the standard error of estimate is 2,36 and the error at the 85 per cent level is 29 per cent. For completeness' sake it was decided to do an analysis on all boreholes with optimum moisture content values less than 13 but excluding te nominal parameters. The first three parameters in the resulting equation are naturally the same as the above ones, but subsequent parameters differ. Because these latter changes do not affect the accuracy appreciably, it is not reported in Appendix D.

(b) Linear regression on optimum moisture content values greater than or equal to 13

A full linear regression analysis was carried out on all data where optimum moisture content is equal to or greater than 13. In the first analysis all parameters were included and the following equation resulted:

$$EMC = 0,21(LL)^{0,7} (\% - 0,425)^{0,3} - 0,015(MDD) + 0,976(RCON) + 30,91 \quad \ldots\ldots \quad (7.11)$$

where RCON = 1 for not failed; 2 for border line and 3 for failed pavements.

This equation has a multiple correlation coefficient of 0,79 and a standard error of estimate of 3,54. The error at the 85 per cent level is 23,0 per cent. This equation obviously only takes the first three parameters into account, as little accuracy results from further refinement. Part of the analysis is reported in Appendix D. For completeness' sake it was also decided to do an analysis on all boreholes with optimum moisture content values greater than or equal to 13 but excluding the nominal and ordinal parameters. The resulting equation is as follows:

$$EMC = 0,25(LL)^{0,7} (\% - 0,425)^{0,3} - 0,018(MDD) - 0,06(\% - 0,075) + 38,5 \quad \ldots\ldots \quad (7.12)$$

This equation has a multiple correlation coefficient of 0,79 and a standard error of estimate of 3,55. The error at the 85 per cent level is 23 per cent. Other useful formulae that may be used if all the best parameters are not available include:

$$EMC = 0,21(LL)^{0,7} (\% - 0,425)^{0,3} - 0,016(MDD) + 33,3 \quad \ldots \quad (7.13)$$

where the multiple correlation coefficient is 0,78, the stand error of estimate is 3,62 and the error at the 85 per cent level is 23 per cent. And:

$$EMC = 0,38(LL)^{0,7} (\% - 0,425)^{0,3} - 1,5 \quad \ldots\ldots \quad (7.14)$$

where the multiple correlation coefficinet is 0,76, the standard error of estimate is 3,76 and the error at the 85 per cent level is 24 per cent.

7.3.4 Combining the equations for the different ranges

Different equations have now been developed for the two intervals the data has been divided into. It is intuitively felt that there should not be an abrupt change of formula at optimum moisture content equal to 13 but rather a gradual change. To achieve this gradual change of formula with optimum moisture content, the following two equations were merged:

$$\text{OMC} < 13: \quad \text{EMC} = 0{,}78(\text{OMC}) - 0{,}156(\text{SE}) + 0{,}54(\text{I}) + 2{,}84 \quad \ldots\ldots \quad (7.8)$$

$$\text{OMC} \geq 13: \quad \text{EMC} = 0{,}21(\text{LL})^{0{,}7}(\% - 0{,}425)^{0{,}3} - 0{,}015(\text{MDD}) + 0{,}976(\text{RCON}) + 30{,}91 \quad \ldots\ldots \quad (7.11)$$

The resulting equation is as follows:

$$\text{EMC} = (-0{,}172(\text{OMC}) + 2{,}676)(\text{OMC}) + (0{,}034(\text{OMC}) - 0{,}535)(\text{SE}) + (-0{,}012(\text{OMC}) + 0{,}185)(\text{I}) + (0{,}046(\text{OMC}) - 0{,}51)(\text{LL})^{0{,}7}(\% - 0{,}425)^{0{,}3} + \text{[illegible]}\text{MC}) + 0{,}036)(\text{MDD}) + (0{,}225(\text{OMC}) - 2{,}37)(\text{RCON}) + \text{[illegible]} - 75{,}1) \quad \ldots\ldots \quad (7.15)$$

The following shows [illegible] prediction accuracy of the two separate equations 7,8 and 7,11 compared with that of the combined single equation 7.15 on identical data for the same range:

	MULTIPLE CORRELATION COEFFICIENTS	
	Two equations	One equation
EMC	0,865	0,871

	STANDARD ERROR OF ESTIMATE	
	Two equations	One equation
EMC	2,79	2,78

It can be seen that the single equation does in fact represent a slight improvement but it does not warrant the added complexity and it is recommended that the two equations be used separately, each for its specific range.

7.3.5 Linear regression using only selected parameters

It is often necessary to predict the equilibrium moisture content if only certain soil test results are available and it was therefore decided to develop models containing only select parameters.

If only the Atterberg limits are available, the following models could be used:

$$EMC = 0,39(LL) \quad \ldots \quad (7.16)$$

This formula has a correlation coefficient of 0,77 and a standard error of estimate of 3,58.

$$EMC = 0,8(PL) + 1,8 \quad \ldots \quad (7.17)$$

This formula has a correlation coefficient of 0,61 and a standard error of estimate of 4,51.

$$EMC = 0,5(PI) + 4,2 \quad \ldots \quad (7.18)$$

This formula has a correlation coefficient of 0,76 and a standard error of estimate of 3,70.

$$EMC = 1,2(LS) + 3,9 \quad \ldots \quad (7.19)$$

This formula has a correlation coefficient of 0,76 and a standard error of estimate of 3,71.

If only the percentage grading analysis results are known:

$$EMC = 0,22(\% - 0,075) + 2,9 \quad \ldots \quad (7.20)$$

This formula has a correlation coefficient of 0,67 and a standard error of estimate of 4,19.

If the grading analysis results and Atterberg limits are known, the following models are useful:

$$EMC = 0,42(LL)^{0,7}(\% - 0,425)^{0,3} - 3,9 \quad \ldots \quad (7.21)$$

This formula has a correlation coefficient of 0,83 and a standard error of estimate of 3,13.

$$EMC = 0,053(LS)(\% - 0,425)^{0,7} + 5,1 \quad \ldots \quad (7.22)$$

This formula has a correlation coefficient of 0,80 and a standard error of estimate of 3,38.

If the compaction characteristics are known:

$$EMC = 0,71(OMC) - 0,011(MDD) + 21,6 \quad \ldots \quad (7.23)$$

This formula has a correlation coefficient of 0,85 and a standard error of estimate of 3,00. Or

$$EMC = 1,1(OMC) - 3,4 \quad \ldots \quad (7.2)$$

This formula has a correlation coefficient of 0,84 and a standard error of estimate of 3,08.

7.3.6 Non-linear regression analysis

From the models developed thus far it is clear that the moisture content may not be predicted sufficiently accurately for design purposes. In an attempt to improve the prediction accuracy, a non-linear regression analysis was carried out, using the Marquardt method of minimizing the sum-of-squares function (Marquardt, 1963).

Much work was done for this analysis and various models were optimized using this technique. Although the prediction accuracies could be increased in some cases, these increases were insignificant (Haupt, 1978) and because the non-linear models are generally more complicated, it was decided to present and recommend for use only the models developed thus far, using conventional techniques.

7.3.7 Treatment of nominal and ordinal parameters

Although nominal and ordinal parameters were analysed in detail in Chapter 5 by a breakdown analysis and although they have been included in many of the equations presented in this chapter, this inclusion is not strictly statistically founded, as pointed out before. In a final effort to try and improve the prediction accuracies of the models, it was decided to use the nominal parameters purely as class dividers and to do the analysis only on the ratio parameters which is statistically correct.

Firstly, analyses were done on all data which were classed according to elevation. It became clear that this parameter on its own does not have a big influence on equilibrium moisture content. In order to determine the combined effect of several parameters, it was decided to consider all cases when elevation was flat and then vary the other parameters. Elevation was thus kept constant and the quality of surface drainage was varied from good to poor, but again no significant effect was observed.

This process was repeated for all nominal and ordinal parameters, but apart from the conclusions drawn in Chapter 5, no parameters indicated clear trends.

Because this investigation, like the breakdown analysis, does not yield trends it was decided to use the equations developed before, regardless of the physical features of the site it is applied to, i.e. they are generally applicable to any set of circumstances and would therefore be conservative in conditions where, say, drainage and geometric design is of a high standard.

7.3.8 Influence of climate

One of the main aims of the moisture investigations undertaken for this study in regions of climatic extremes was to determine whether the models developed for the Transvaal could be extended to other climatic regions.

The first step in establishing whether this could be done was to combine the best predictors with climatic data and to repeat the regression analysis. Equation (7.7) had been developed for all regions but not combining the predictors, while equation (7.24) was developed for all regions but combining the mean annual rainfall with the variables:

$$EMC = 0,67(OMC) + 0,18(TL)^{0,7}(\% - 0,425)^{0,3} - 4,2 \quad .... \quad (7.7)$$

$$EMC = 0,73(OMC) + 0,003(MAR)(LS)(\% - 0,425)^{0,7} - 1 \quad .... \quad (7.24)$$

In order to determine the approximate prediction accuracies of these models, the standard error of estimate had to be calculated, as both the correlation coefficient and the standard deviation are influenced by the amount of variation of the criterion and cannot be compared across situations where the criterion has different standard deviations. The standard error of estimate takes this into account and may thus be compared across unequal amounts of variation in the criterion. The prediction accuracies for the different regions are compared in Table 7.3 and it is clear that there is no change in prediction accuracy when the predictors were combined with climatic indices. This simply means that one cannot improve the accuracy of prediction by including climatic data, but it does not conclusively show that the same techniques can be used in all areas.

TABLE 7.3: COMPARISON OF PREDICTION ACCURACIES

Region		r	S_y	$S_{yx}=S_y\sqrt{(1-R^2)}$	$\bar{X}$	Accuracy at 85% level (%)
Tvl	Eq. 7.7	0,848	5,53	2,93	10,5	28
	Eq. 7.24	0,846	5,53	2,95	10,5	28
Cape	Eq. 7.7	0,906	10,76	4,55	16,8	27
	Eq. 7.24	0,907	10,78	4,54	16,8	27
SWA	Eq. 7.7	0,391	2,37	2,18	5,7	38
	Eq. 7.24	0,444	2,37	2,12	5,7	37

It was next decided to check the best prediction equation for Transvaal conditions, applied to the George and Keetmanshoop areas. Equation (7.1) was checked for all regions:

$$EMC = 0,79(OMC) + 15,92(CBR)^{-0,6} - 3,0 \quad \ldots\ldots\ldots\ldots \quad (7.1)$$

The correlation coefficients compared as follows:

Transvaal: 0,86; Cape: 0,83; SWA: 0,62.

The correlation coefficients were tested for similarity using the Fisher transformation (Downie and Heath, 1970) and it was found that the Transvaal formula is not applicable to the SWA conditions. It could, however, not conclusively be proven that the Transvaal model may be applied to Cape conditions unaltered because so few data points are available from the Cape. In practical terms, because it could not be applied to Cape conditions, it is reasonable to assume, in the absence of more information, that it is applicable. In order to bear this out it was decided to do independent regression analyses on each region separately and to compare the results. It was decided not to include more than two predictors in any model, because so few data are available for both the Cape and SWA.

The Cape data yielded similar prediction models with nearly identical accuracies and it confirms the assumption that the Transvaal models may be used in wet areas of southern Africa. The best model for George is:

$$EMC = 1,4(OMC) - 4,1 \quad \ldots\ldots \quad 27\text{ \% accuracy} \quad \ldots\ldots\ldots\ldots \quad (7.25)$$

(cf EMC = 1,1(OMC) - 3,4 ..... 28 % accuracy for the Transvaal)

This regression equation (equation 7.25) had been developed from only 20 results because too little material had been sampled from the other boreholes. In order to increase the accuracy of the equation, the missing values of OMC were estimated using relationships between OMC and available test results. This could not be done successfully because too little information is available about the best predictors of OMC and so the missing values of OMC could not be predicted sufficiently accurately.

The best prediction model developed for SWA was totally different from the models developed in the Transvaal:

EMC = 1,1(RCON) - 0,08(CBR) + 6,6 ..... 25 % accuracy ..... (7.26)

More accurate prediction formulae cannot be developed for SWA because too little information is available. Equation (7.26) was based on only 28 results and is thus not reliable or of much practical significance. General guidelines for dry regions (MAR < 200 mm) are:

(a) The only parameter that correlates reasonably well with EMC is CBR.

(b) Physical conditions such as the condition of the road, the degree of rutting and surface permeability are better related to the moisture conditions than are the material properties.

(c) A rough estimation of the equilibrium moisture content when the physical conditions are favourable is:

EMC < 0,66(OMC)

Approximately ninety per cent of local values fall in this range. Work is in progress to accumulate more information about SWA so that a complete separate analysis may be carried out in the dry regions to firstly establish at what climatic conditions the trends of the mild and wet climates cease to be applicable and, secondly, to establish relevant relationships for the dry regions.

To summarize, it can be said that the models developes here are applicable where the mean annual rainfall lies between 400 - 1 500 mm, Thornthwaite's Index between -45 and +100 and Weinert's N-value between 1 and 7,6. These models have been shown to be over-conservative in drier regions but too few data are available to develope separate models for the drier regions at present. For the climatic regions given above, the moisture condition in the subgrade of a covered area is relatively insensitive to the climate although there is a trend for the models to over-predict the moisture content in the drier regions and to under-predict in the wetter regions, but this trend is not significant.

7.3.9 Influence of compactive effort

Because it is customary in the Transvaal to test the subgrade material at Proctor and not Mod. AASHTO compaction, it was decided to use the former compaction effort for this investigation. Subsequently a laboratory investigation was carried out to determine the relationship between Proctor and Mod. AASHTO compaction because Mod. AASHTO is the standard used in the rest of South Africa. It is outside the scope of this dissertation to report in detail about this analysis, which is still in progress, but the initial results are summarized in Table 7.4. It must be pointed out that the results of 783 samples were analysed, which means that (a) a wide variety of material types was covered, and (b) the results are statistically significant.

TABLE 7.4: SUMMARY OF RELATIONSHIPS BETWEEN PROCTOR AND MOD. AASHTO COMPACTION

Proctor characteristics	r	Error at 85% level
$OMC = 1,25\ (OMC_M) - 0,5$	0,92	10%
$MDD = 1,2\ (MDD_M) - 533$	0,93	2%

It is clear from the Table that the Proctor and Mod. AASHTO compaction characteristics correlate highly and that one may be deduced from the other with a fair amount of accuracy.

These relationships make it possible to use the moisture prediction equations if only the Mod. AASHTO compaction characteristics are available, by simple substitution. The accuracy of the moisture prediction equations will obviously be slightly reduced if these conversions are used.

7.3.10 Point of Diminishing Returns

From all the data presented in the text and in the appendices, it was concluded that the point of diminishing returns had been reached in this analysis. Most applicable statistical procedures had been tried and little extra accuracy was likely to result except with much extra effort. The reasons for deciding that this was the point of diminishing returns, may be summarized as follows:

(a) All available parameters have been analysed and transformed into linear relationships.

(b) Linear regressions have been performed on all the available data.

(c) Linear regressions have been performed on certain ranges of data.

(d) Equations resulting from these linear regressions on certain ranges have been combined.

(e) Non-linear regressions have been performed on certain samples of data.

(f) Some of the parameters used in the analysis have inherent errors in their method of measurement, as stated before, and some of the scatter reported here as due to the lack of prediction accuracy is in fact due to these errors. One could get an idea of how good the prediction equations are and whether more predictors are required or whether the available ones should be determined more accurately, if information about the reliability of the predictors were available. This kind of data is not available for all the parameters used in this investigation but Mitchell and Smith (1974) have analysed four of the parameters, viz. LL, PI, percentage material finer than 0,075 mm (% - 0,075) and percentage material finer than 0,425 mm (% - 0,425). Various statistical parameters are reported in their paper and from these, correlation coefficients required to check this can be calculated.

The way to check whether new predictors need to be involved in the prediction model is to adjust the multiple correlation coefficient to exclude the errors in the estimators. This is, however, only a rough adjustment:

$$r_{ad} = \frac{r_{OB}}{\sqrt{r_{PI}}\sqrt{r_{LL}}\sqrt{r_{(\%-0,075)}}\sqrt{r_{EMC}}} \quad \ldots\ldots\ldots\ldots\ldots \quad (7.27)$$

where r_{ad} = adjuated multiple correlation coefficient

r_{OB} = multiple correlation coefficient of prediction equation

r_{PI} = correlation coefficient of PI determinations

r_{LL} = correlation coefficient of LL determinations

$r_{(\%-0,075)}$ = correlation coefficient of (%-0,075) determinations

r_{EMC} = correlation coefficient of EMC determinations.

Assuming that the repeatability of testing in this case had been the same as that of the reported case, the adjusted coefficient becomes greater than unity which means that firstly our material testing was done with a higher measure of repeatability than those in the reported case and secondly that the prediction equations cannot be improved significantly with this kind of data.

It was therefore decided not to continue with sophisticated statistical analyses because it was apparent at this stage that the nature of the predictors would render *this exercise futile. It is concluded that* no better prediction equation can be developed empirically and that for better accuracy, theoretical methods should be pursued. It must be noted that if the prediction parameters are more carefully determined in practice, these equations will yield answers with accuracies superior to those reported here.

7.3.11 Summary of Practical Prediction Formulae

The most important moisture prediction formulae may be summarized as in Table 7.5.

TABLE 7.5: PRACTICAL PREDICTION FORMULAE

Predicted equilibrium moisture content	% error at level (%)		
	85	90	95
For OMC < 10, PEMC =			
$1,09(\% - 0,075) + 3,8$	31	38	49
For OMC < 13, PEMC =			
*$1,1(OMC) - 3,4$	31	38	49
$1,38(OMC) - 3,9$			
$0,86(OMC) + 0,07(\% - 0,075) -$ [illegible]	29	36	46
$1,08(OMC_M) + 0,07(\% - 0,075) - 3,4$			
$0,87(OMC) - 0,17(SE) + 1,7$	29	36	46
$1,08(OMC_M) - 0,17(SE) - 2,1$			
For OMC ≥ 13, PEMC =			
$0,38(LL)^{0,7}(\% - 0,425)^{0,3} - 1,5$	24	[illegible]	38
$0,21(LL)^{0,7}(\% - 0,425)^{0,3} - 0,016(MDD) + 33,3$	23	[illegible]	36
$0,21(LL)^{0,7}(\% - 0,425)^{0,3} - 0,019(MDD_M) + 41,8$			
All values of OMC, PEMC =			
$0,053(LS)(\% - 0,425)^{0,7} + 5,1$	30	37	47
$0,42(LL)^{0,7}(\%$, 0,4[illegible]5$)^{0,3} - 3,9$	28	34	44
$1,1(OMC) - 3,4$	28	34	44
$0,71(OMC) - 0,011(MDD) + 21,6$	27	33	43
$0,89(OMC_M) - 0,013(MDD_M) + 27,1$			
$0,6(OMC) + 0,2(LL)^{0,7}(\% - 0,425)^{0,3} - 4,2$	27	33	43
$0,75(OMC_M) + 0,2(LL)^{0,7}(\% - 0,425)^{0,3} - 4,5$			
Other useful relations (95 % C.I. upper limits)			
PEMC $\leq 0,47(LL)^{0,7}(\% - 0,425)^{0,3} - 2,9$			
PEMC $\leq 0,52(LL) + 1,5$			
PEMC $\leq 1,36(OMC) - 2,3$			
PEMC $\leq 1,7(OMC_M) - 3,0$			

* This formula is slightly less accurate when applied to the range $OMC_P \leq 10$

7.4 The prediction of CBR

7.4.1 Earlier work

It has for many years been tried to predict the CBR of soils from the plasticity characteristics.

Evans (1951) related the CBR to the Vicksburg penetrometer readings. Nascimento and Simoes (1957) found that for soft materials the elastic modulus is 10 to 20 times the CBR value and the modulus of subgrade reaction is 1/8 to 1/4 of the CBR value, and for hard materials these are 10 to 30 and 1/8 to 1/3 respectively. Robinson (1958) developed the following relation, on the basis of load on a 3 inch diameter plate at failure of the soils:

$$\log CBR = 0{,}76837 \log (\text{Plate load in lbs}) - 1{,}64420 \ldots \quad (7.28)$$

In 1961 Wiseman and Zeitlen tried to correlate the in-situ CBR with in-situ shear strength by a static penetration test. In the same year Black published his first papers on work he was doing to predict the CBR from plasticity data. This led to the publication of a practical method in 1962 for doing just this. The method proposes that a first approximate of the CBR may be deduced from estimating the bearing capacity of the soil, which can be deduced from a knowledge of its suction and true angle of friction. It also shows how the suction and true angle of friction may be inferred from a knowledge of the liquid and plastic limits and moisture content of the soil. This means that the CBR and its variation with moisture content may be estimated from the results of various plasticity tests. The method is only applicable to cohesive soils. Although this method is valuable, practical and complete, it is based on relationships between suction, moisture content and Atterberg limits which are not validated for local soils. A need was therefore felt to develop general relationships between CBR and Atterberg limits for local conditions. For completeness' sake the three figures Black used in his method are included as they may be helpful under certain circumstances. Figure 7.9 depicts a general relationship between moisture content and suction for different soils. Figure 7.10 shows how the consistency index $(\frac{LL - W}{PI})$ may be determined from a knowledge of suction and plasticity (W = sample moisture content). Finally Figure 7.11 shows how the CBR may be determined from the consistency index and plasticity index.

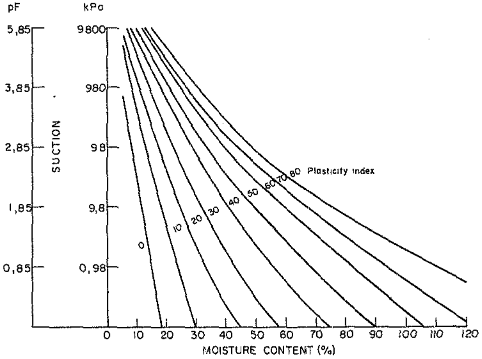

FIGURE 7.9

THE RELATION BETWEEN SUCTION AND MOISTURE CONTENT AT VARIOUS PLASTICITY INDICES (BLACK, 1962)

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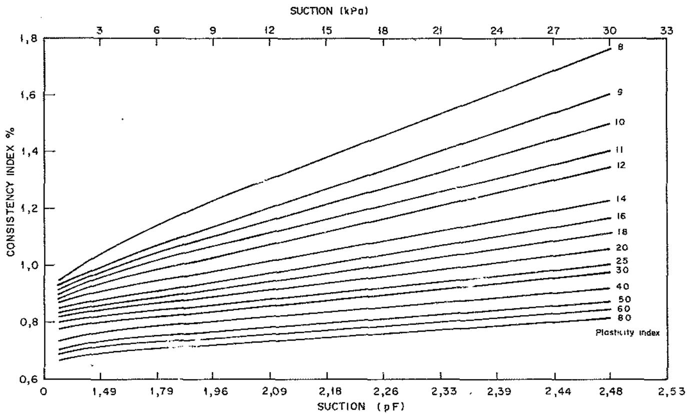

FIGURE 7.10

THE VARIATION OF CONSISTENCY INDEX WITH SUCTION AT DIFFERENT PLASTICITY INDICES (BLACK, 1962)

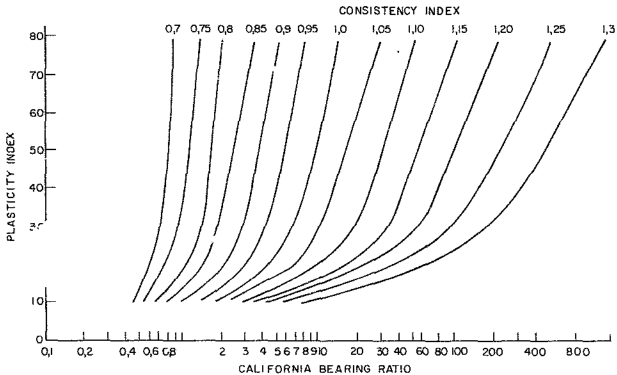

FIGURE 7.11

RELATION BETWEEN C.B.R. AND PLASTICITY INDEX AT VARIOUS CONSISTENCY INDICES (BLACK, 1962)

In 1970 Agarwal and Ghanekar developed a model with which the CBR could be predicted from the OMC and LL, viz. (compaction effort not known assumed British 2,5 kg rammer method):

$$CBR = 21,2786 - 16,2921 \log (OMC) + 0,0696(LL) \quad \ldots\ldots \quad (7.29)$$

The multiple correlation coefficient of this model is 0,58 and the standard error of estimate is 1,8. They also reported the following less accurate models:

$$CBR = 20,2809 - 14,31281 \log (OMC) + 0,0745(PI) \quad \ldots\ldots \quad (7.30)$$

$$CBR = 18,735 - 12,861 \log (OMC) + 0,052(PL) \quad \ldots\ldots \quad (7.31)$$

These models clearly are only of limited practical value because

(a) they were developed from only 48 samples;

(b) the CBR of samples used only varied between 1 and 9; and

(c) the prediction accuracy is very low.

In 1975 Livneh and Ishai also proposed a method to predict the CBR from a knowledge of PL, moisture content and degree of saturation. From Figure 7.12 an estimate of the suction in the sample at the measured moisture content may be obtained. This method estimates the CBR at this moisture content and not saturation. This suction is corrected for partial saturation by applying equation (7.32):

$$\tilde{\phi} = \phi S_r^{\,2} \quad \ldots\ldots \quad (7.32)$$

where $\tilde{\phi}$ = effective suction

ϕ = suction obtained from Figure 7.12

S_r = degree of saturation.

From Figure 7.13 an estimate of the in-situ CBR for cohesive materials may now be made.

Gawith and Perrin (1962) suggest the following equations for the evaluation of the CBR values:

$$\log CBR = 1,886 - 0,0143D - 0,00045A + 0,00515\frac{B}{A} - 0,0000456\left(\frac{B}{A}\right)^2 - 0,0037E \quad \ldots\ldots \quad (7.33)$$

where A = percentage finer than 0,425 mm

B = percentage finer than 0,075 mm

D = plasticity index

E = percentage finer than 2,36 mm

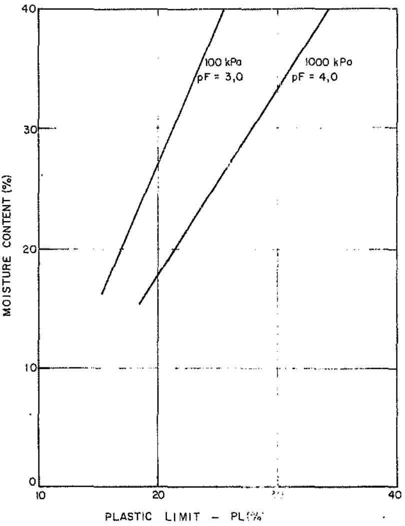

FIGURE 7.12

RELATIONSHIP BETWEEN PLASTIC LIMIT, MOISTURE CONTENT AND SUCTION (Livneh et al, 1967)

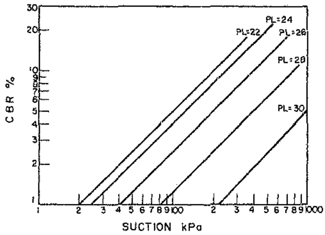

FIGURE 7.13

RELATIONSHIP BETWEEN SUCTION, CBR AND PLASTIC LIMIT (Livneh and Ishai, 1975)

$$CBR = 4,5 + \frac{(20 - G)^2}{18} \quad \text{.............................} \quad (7.34)$$

where G = group index according to the AASHTO soil classification system.

There are numerous more empirical models available to predict CBR (Wermers, 1963) but most of them are limited to certain soil types and none of them has won general acceptance. See also U S Army Corps of Engineers (1958), Packard (1973), Gawith and Perrin (1962), Road Note 29, Sharma and Sharma (1964), Livneh and Greenstein (1975).

7.4.2 The analysis of local data

It should be borne in mind that local prediction models are based on subgrade and subbase materials only and may not indiscriminately be applied to better quality materials. It is also very important to remember that the CBR values predicted with these models will be slightly higher than a soaked CBR value for Proctor compaction because they were penetrated dry of optimum. More work is at present being done to relate CBR quantitatively to compactive effort and moisture content during testing.

When analysing the local data it is immediately clear why most of the existing models are non-linear. This is because the CBR does not correlate with the soil parameters in a linear model. Non-linear logarithmic or exponential models explain significantly more variation. It is also very interesting to note that the CBR correlates significantly better with grading parameters (specifically %-0,075 mm) than with plasticity or compaction parameters (with the exception of bar linear shrinkage).

Table 7.6 indicates the correlation coefficients between CBR and various other parameters.

TABLE 7.6: CORRELATION COEFFICIENTS BETWEEN CBR AND OTHER PARAMETERS

Linear		Non-linear	
	CBR		CBR
%-0,075	-0,69	log(LST)	-0,78
SE	+0,69	log(%-0,075)	-0,76
GM	+0,64	log(PIT)	-0,73
LLT	-0,64	log(LLT)	-0,72
MDD	+0,62	e^{SE}	+0,69
LST	-0,62	MDD^2	+0,65
LS	-0,58	log(LS)	-0,64
PIT	-0,58	e^{GM}	+0,64
%-0,425	-0,58	log(OMC)	-0,61
OMC	-0,55	log(PI)	-0,60
PI	-0,52	log(%-0,425)	-0,58
LL	-0,47	log(LL)	-0,51

From this table it can be seen that the logarithm of the transformed linear shrinkage is the best predictor of CBR for subgrade soils (i.e. relatively low CBR materials), and the following formula should be used:

$$CBR = 80{,}5 - 32{,}0\log\{(LS)\cdot(\%\text{-}0{,}425)^{0{,}7}\} \qquad (7.35)$$

The correlation coefficient of this model is 0,77 and the standard error of estimate is 8,68. The prediction accuracy at the 85 per cent level is 48 per cent.

A linear and non-linear regression analysis was carried out and Table 7.7 summarizes the resulting prediction models.

TABLE 7.7: SUMMARY OF USEFUL CBR PREDICTION MODELS

CBR =	Accuracy at 85 % level		
$2,1(e^{GM}) - 23 \log \{LS\ (\%-0,425)^{0,7}\} + 54$	41%	$r = 0,83$	$\sigma = 7,6$
$96,3 - 17,8 \log \{(LS)(\%-0,425)^{0,7}\} - 28,7 \log (\%-0,075)$	42%	$r = 0,83$	$\sigma = 8,5$
$97,7 - 17,1 \log \{(PI)(\%-0,425)^{0,5}\} - 30,7 \log (\%-0,075)$	44%	$r = 0,81$	$\sigma = 8,0$
$119,6 - 33 \log \{(LL)^{0,7}\ (\%-0,425)^{0,3}\} - 33,2 \log (\%-0,075)$	45%	$r = 0,79$	$\sigma = 8,4$
$80,5 - 32,3 \log \{(LS)(\%-0,425)^{0,7}\}$	48%	$r = 0,77$	$\sigma = 8,7$
$90 - 47,4 \log (\%-0,075)$	48%	$r = 0,77$	$\sigma = 8,7$

It should be borne in mind that these formulae were developed for subgrade materials and significantly underestimate the CBR values of materials with CBR values of more than 50. Secondly it must be remembered that this constitutes a preliminary investigation and more work should be done if more accurate prediction techniques are required.

7.5 Prediction of compaction characteristics

7.5.1 Earlier work

In 1938 Woods and Litchiser, after analysing 1 367 Ohio soils, showed the general interrelation of plastic limit, plasticity index, liquid limit, optimum moisture content and maximum dry density. In 1955 Rollins and Davidson related Atterberg limits to suctions. Figure 7.14 (Croney, 1977) shows the relationship between dry density, moisture content and air voids.

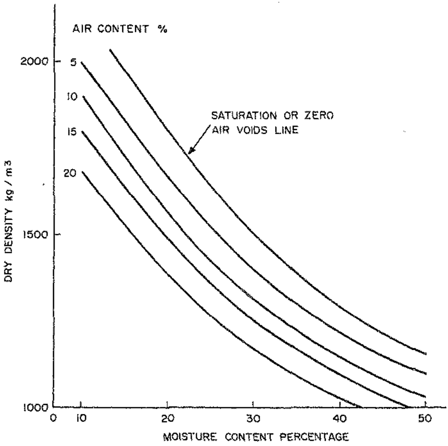

FIGURE 7.14

RELATIONSHIP BETWEEN DRY DENSITY, MOISTURE CONTENT, AND PERCENTAGE AIR CONTENT (CRONEY, 1977)

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In 1959 Yoder published a linear relationship between dry density and the log of energy input. He also presented a relationship between Std. AASHTO and Mod. AASHTO dry densities. Livneh and Ishai (1977) do not agree with his line but found a similar trend, as shown in Figure 7.15. Yoder (1959) also indicated a relationship between maximum dry density and compactive energy as shown in Figure 7.16, while Livneh and Greenstein (1978) presented a generalised relationship between optimum moisture content and maximum dry density for <u>any</u> effort, as shown in Figure 7.17.

Spangler (1951) suggested the following relationship between Std. Proctor density and OMC:

$$MDD = \frac{6250K_1}{S_L\left(\frac{\% - 4,67}{\% - 0,425} - 1\right) + \frac{100}{S_R}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7.36)$$

$$OMC = S_L\frac{(\% - 0,425)}{(\% - 4,67)} + K_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7.37)$$

where S_L = shrinkage limit

S_R = shrinkage ratio

%-4,67 = percentage finer than 4,67 mm

%-0,425 = percentage finer than 0,425 mm

$K_1 = \frac{104 - 0,67PI}{100}$

$K_2 = 0,33PI - 4$

PI = plasticity index

Jumikis (1958) presented a chart relating optimum moisture content obtained from "standard soil compaction tests" with liquid limit and plasticity index from various New Jersey glacial soils.

In 1948 Rowan and Graham presented the following formulae for estimating the maximum dry density and optimum moisture content for Proctor compaction:

$$\text{Maximum dry density} = \frac{D}{1 + \frac{D - C}{1\,000S}} \quad (kg/m^3) \quad \ldots\ldots\ldots\ldots (7.38)$$

$$\text{Optimum moisture content} = S_L\left(\frac{B}{A}\right) \quad (\%) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7.39)$$

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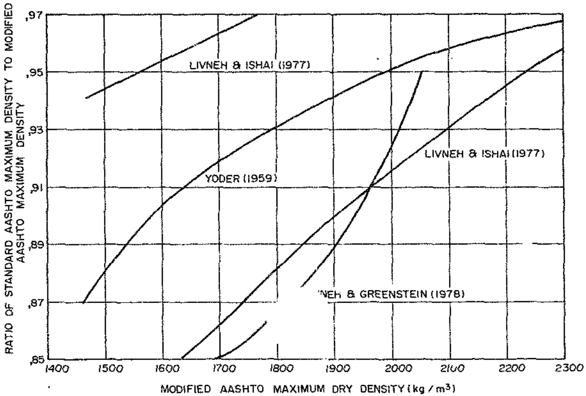

FIGURE 7.15

RELATIONSHIP BETWEEN STANDARD AASHTO AND MODIFIED AASHTO DENSITIES

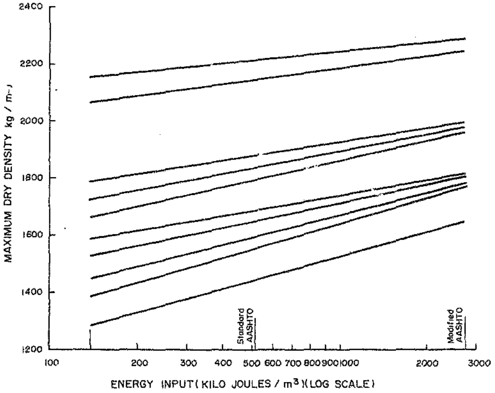

FIGURE 7.16

VARIATION OF MAXIMUM DENSITY WITH COMPACTIVE EFFORT FOR DIFFERENT SOILS (YODER, 1959)

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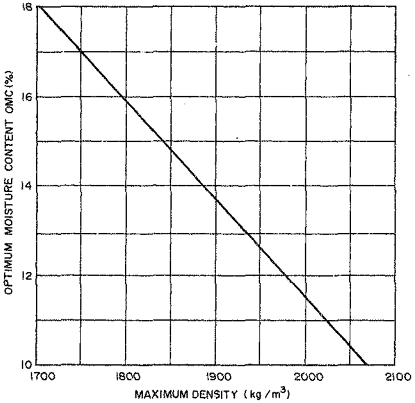


FIGURE 7.17

RELATIONSHIP BETWEEN OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY FOR ANY COMPACTIVE EFFORT (LIVNEH & GREENSTEIN, 1978) where $D = \frac{CA}{B}$

A = percentage finer than 4,67 mm

B = percentage finer than 0,425 mm

C = 1 000 x shrinkage ratio (kg/m^3)

Davidson and Gardiner (1949) adjusted these formulae to fit the data from 210 soils from 11 states better:

$$\text{Maximum dry density} = \frac{K_1 \cdot 10^5}{S_L\left(\frac{B}{A} - 1\right) + \frac{100}{R}} \quad \ldots\ldots\ldots\ldots \quad (7.40)$$

$$\text{Optimum moisture content} = S_L\left(\frac{B}{A}\right) + K_2 \quad \ldots\ldots\ldots\ldots \quad (7.41)$$

where $K_1 = \frac{312 - 2(PI)}{300}$

PI = plasticity index

R = shrinkage ratio

$K_2 = \frac{PI}{3} - 4$

Turnbull (1948) presented a method using the soil gradation to estimate the optimum moisture content.

In 1958 the Bureau of Public Roads (Yemington) correlated optimum moisture content and maximum dry density with classification data of 972 soil samples from 31 states. The compaction effort for this study was according to AASHTO Designation T99-49, which is uncommon in South Africa, and the results of this study is thus not represented here.

In 1961 the Bureau of Public Roads (Ring et al, 1962) carried out a second study to improve the prediction methods developed from the first one. Ring et al reported several prediction models that resulted from the analysis together with their associated accuracies. They did, however, not suggest generalized equations but developed different formulae for different regions.

7.5.2 The analysis of local data

The aim of this investigation is to develop relationships between the Atterberg limits and compaction characteristics. If only the Atterberg limits and the grading of a material is known, a very good approximation of the compaction parameters can be made, using the equations developed below. It must be remembered that the models have been developed largely from subgrade quality materials and they may not be extrapolated to better quality materials without some loss in prediction accuracy. It is also important to bear in mind that these relationships were developed using only Proctor energy. The same relationships may not be applicable to other energy levels.

7.5.2.1 Prediction of Proctor maximum dry density

If only the Atterberg limits and the percentage finer than 0,425 mm is known, a fairly good approximation of the Proctor maximum dry density may be made:

$$MDD = 10{,}3(PI) - 20{,}5(LL)^{0{,}7}(\%\text{-}0{,}425)^{0{,}3} + 2477 \;\ldots\ldots\; (7.42)$$

This model has a standard error of estimate of 66 and a multiple correlation coefficient of 0,93 and the error at the 85 per cent level is 3,5 per cent.

In order to produce a curve relating Atterberg limits to compaction characteristics, the following formula was developed:

$$MDD = 0{,}83(PL) - 14(LL)^{0{,}7}(\%\text{-}0{,}425)^{0{,}3} + 2364 \;\ldots\ldots\; (7.43)$$

This model has a multiple correlation coefficient of 0,91 and a standard error of estimate of 74. The error at the 85 per cent level is 3,94.

7.5.2.2 Prediction of Proctor optimum moisture content

The following equation may be used to predict the Proctor optimum moisture content from Atterberg limits and grading information:

$$OMC = 0{,}3(LL)^{0{,}7}(\%\text{-}0{,}425)^{0{,}3} + 0{,}2(PL) - 1 \;\ldots\ldots\; (7.44)$$

This formula has a multiple correlation coefficient of 0,93 and a standard error of estimate of 1,59. The error at the 85 per cent level is 11,9 per cent.

It is also interesting to note that there is a fairly good relationship between Proctor OMC and maximum density:

$$OMC = 57,5 - 0,024(MDD) \quad \ldots\ldots (7.45)$$

This model has a standard error of estimate of 1,73 and a correlation coefficient of 0,92. The error at the 85 per cent level is 13 per cent.

Figure 7.18 shows the relationship between consistency indices and compaction parameters for local soils. It is also useful to know how the compaction parameters at different energies relate to each other. The following relationships exist for local subgrade soils. Preliminary work regarding similar relationships for other pavement layers has been started, but qualitative results are not available yet:

$$OMC = 1,25(OMC_M) - 0,5 \quad \ldots\ldots (7.46)$$

$$MDD = 1,2(MDD_M) - 533 \quad \ldots\ldots (7.47)$$

where OMC = Proctor optimum moisture content
OMC_M = modified AASHTO optimum moisture content
MDD = Proctor maximum dry density
MDD_M = modified AASHTO maximum dry density.

Equation (7.46) has a correlation coefficient of 0,92 and the error at the 85 per cent level is 10 per cent, while equation (7.47) has a correlation coefficient of 0,93 and the error at the 85 per cent level is 2 per cent. Seven hundred and eighty-three samples were used in developing these equations.

As can be seen from Figure 7.15, there does not exist a definite relationship between the ratio of the compactive efforts and the density. This is also the case for local soils. Other factors influence the relationship and there is no way any of the lines drawn in Figure 7.15 may be applied to local conditions. More work has to be done in this regard. Figure 7.19 shows the situation for local conditions.

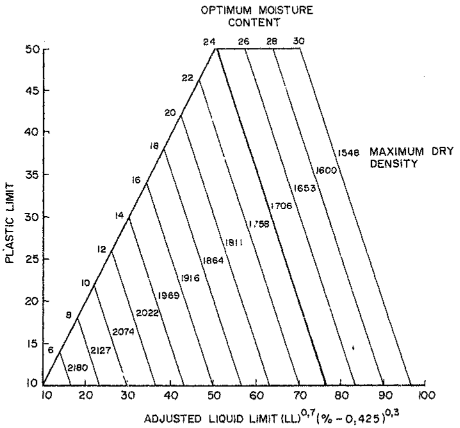
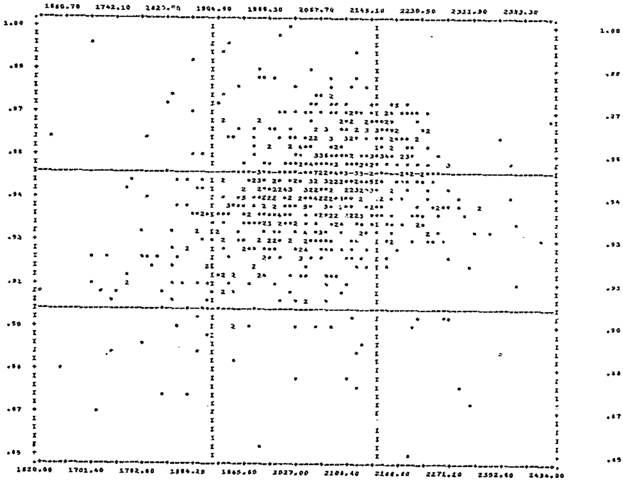


FIGURE 7.18

RELATIONSHIP BETWEEN PROCTOR COMPACTION CHARACTERISTICS AND CONSISTENCY INDICES

FIGURE 7.19

SCATTERGRAM OF (DOWN) Std. AASHTO density/Mod. AASHTO density (ACROSS) Mod. AASHTO density



7.6 *Relationships between various Atterberg limits*

The aim of this section is not to predict the Atterberg limits from other soil constants, but to show how the Atterberg limits are interrelated.

7.6.1 Earlier work

In 1967 Van Rooyen suggested the folliwng interparameter correlations:

$$LS = 0{,}46(PI) \quad (r = 0{,}90) \quad \ldots \quad (7.48)$$

$$PI = 0{,}58(LL) - 4{,}46 \quad (r = 0{,}85) \quad \ldots \quad (7.49)$$

In 1978 Livneh and Greenstein found that for Israeli loess and silty clays PI could be predicted from a knowledge of LL alone:

$$PI = 0{,}75(LL) - 9{,}26 \quad \ldots \quad (7{,}50)$$

They also found that the OMC (for any energy effort) is related to the maximum dry density as follows:

$$OMC_A = 55 - 0{,}022(MDD_A) \quad \ldots \quad (7.51)$$

Ramiah et al (1970) proposed that the Std. AASHTO maximum dry density may be estimated as follows:

$$MDD_S = 2125 - 10(LL) \quad \ldots \quad (7.52)$$

The Std. AASHTO optimum moisture content may be estimated as follows:

$$OMC_S = \frac{1}{3}(LL + 15) \quad \ldots \quad (7.53)$$

7.6.2 Analysis of local data

7.6.2.1 Liquid limit as independent variable

The best predictor of liquid limit is linear shrinkage:

$$LL = 2{,}98(LS) + 11{,}4 \quad \text{.................................} \quad (7{,}54)$$

This formula has a correlation coefficient of 0,94 and a standard error of estimate of 3,82. The error at the 85 per cent level is 13 per cent.

To increase the prediction accuracy the following equation may be used:

$$LL = 2{,}23(LS) + 0{,}97(PL) \quad \text{.............................} \quad (7{,}55)$$

In this case the correlation coefficient is 0,99 and the standard error of estimate is 1,69. The error at the 85 per cent level is 5 per cent.

7.6.2.2 Plastic limit as independent variable

Liquid limit is the best predictor of plastic limit:

$$PL = 0{,}3(LL) + 6{,}8 \quad \text{.................................} \quad (7{,}56)$$

This formula has a correlation coefficient of 0,82 and the standard error of estimate is 2,5. The error at the 85 per cent level is 15 per cent. If the bar linear shrinkage is also known, the following model may be used:

$$PL = 0{,}83(LL) - 1{,}69(LS) + 2{,}2 \quad \text{....................} \quad (7{,}57)$$

In this case the correlation coefficient is 0,94 and the standard error of estimate is 1,5. The error at the 85 per cent level is 9 per cent.

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In this case the correlation coefficient is 0,94 and the standard error of estimate is 1,5. The error at the 85 per cent level is 9 per cent.

7.6.2.3 Plasticity index as independent variable

Bar linear shrinkage is the best predictor of PI:

$$PI = 2,2(LS) - 0,2 \quad \ldots\ldots (7,58)$$

This formula has a correlation coefficient of 0,98 and a standard error of estimate of 1,69. The error at the 85 per cent level is 13 per cent. If LL is also known, the following equation may be used:

$$PI = 1,7(LS) + 0,17(LL) - 2,2 \quad \ldots\ldots (7,59)$$

where the correlation coefficient is 0,98 and the standard error of estimate is 1,55. The error at the 85 per cent level is 11,5 per cent.

7.6.2.4 Bar linear shrinkage as independent variable

As indicated in section 7.6.2.3 the bar linear shrinkage is closely related to PI and little change in prediction accuracy will result from including more predictors.

7.7 Conclusions

The main conclusions drawn from this chapter may be summarized as follows:

(a) Although some non-standard statistical procedures were employed in this analysis, they had all been used successfully previously.

(b) It has conclusively been shown that the Atterberg limits adjusted for the percentage material finer than 0,425 mm correlates significantly better with the subgrade moisture conditions than the unadjusted Atterberg limits. Indications are that the adjusted Atterberg limits also correlate better with other soil parameters such as heave potential and CBR. More work needs to be done to develop more detailed adjustments regarding specific properties but the provisional general adjustment proposed is:

$$AAL = (AL)^{0,7} \; (\%\text{-}0{,}425)^{0,3}$$

where

AAL = adjusted Atterberg limit or linear shrinkage

AL = Atterberg limit or linear shrinkage

%-0,425 = percentage finer than 0,425 mm.

(c) It has conclusively been shown that the Atterberg limits do not change significantly with climate.

(d) The optimum moisture content is the best predictor of moisture conditions where the optimum moisture content is less than 13, but the adjusted liquid limit is the best predictor where optimum moisture content is greater than 13.

(e) It is suggested that two separate prediction equations be used depending on whether the optimum moisture content is above or below 13, seeing that a combination of the equations does not produce significantly higher accuracies.

(f) A comprehensive non-linear regression analysis was carried out without much success, chiefly because the best predictors have in fact linear relationships with the subgrade moisture content.

(g) Climate was found not to have a significant influence on the subgrade moisture conditions as long as the mean annual rainfall lies between 400-1 500 mm. In dry climates (MAR<200 mm) the prediction equations developed in this chapter are not applicable as they over-predict the moisture conditions. More work should be done in this regard.

(h) Although the compactive characteristics used in this analysis had been done at Proctor compaction, accurate relationships between Proctor and Mod. AASHTO characteristics have also been developed making the prediction equations applicable to the latter compaction effort by using the relevant substitutes.

(i) It is felt that more accurate empirical prediction techniques cannot be developed for local conditions because of the size of the sample analysed and the nature of the data.

(j) Relationships between cheap soil tests and soil engineering properties like CBR and compaction characteristics have also been developed so that less testing may be necessary under certain circumstances.

Problem materials may also easily be identified in this way. This was a preliminary study on mostly subgrade quality material, and more work should be done in order to increase the practical applicability to all pavement layers. The proposed formulae may be used for (i) determining the amount of water to use in the compaction test for the first moisture density point, (ii) rapidly appraising compaction test results, (iii) reducing the number of compaction tests required, and (iv) denoting unusual soils that are different from those generally encountered and which may cause construction difficulties.

CHAPTER 8

DEVELOPMENT OF A RATIONAL MOISTURE PREDICTION TECHNIQUE FOR SOUTHERN AFRICA

CHAPTER 8

DEVELOPMENT OF A RATIONAL MOISTURE PREDICTION TECHNIQUE FOR SOUTHERN AFRICA

CONTENTS

CHAPTER 8

DEVELOPMENT OF A RATIONAL MOISTURE PREDICTION TECHNIQUE FOR SOUTHERN AFRICA

8.1 Introduction

Fairly reliable empirical moisture prediction techniques have been developed in Chapter 7. The best accuracy that could be achieved at the 85 per cent level is approximately 78 per cent.

This would usually be very helpful in most engineering situations, but because some problem soils are so sensitive to small moisture changes, a need was felt for even more accurate models. A great deal of effort was put into refining the empirical models, but the reliability could not be improved significantly. The OECD (1973), after a thorough literature survey, found that rational prediction techniques may result in better prediction accuracies. Theoretical models also showed promise with the rapid development of digital computers, but seeing that the input parameters are so variable, it was decided to carry out a pilot study to establish the merits of a rational approach.

No attempt had been made in this country to use suction as a basis for predicting a design moisture content, although this approach was well established overseas (Russam and Coleman, 1961). As a first approximation it was decided to validate the reported relation between climate and equilibrium suction for southern African conditions and to produce a map showing equilibrium suction contours for the subcontinent. If this could be done successfully, it would virtually establish a basis for a rational prediction technique, because such a map used in conjunction with a soil moisture characteristic curve provides a method for establishing a design moisture content.

It is realised that very few laboratories in South Africa are at present in a position to determine soil moisture characteristic curves as a matter of routine. It is also true that much work has to be done to properly contour equilibrium suction lines for southern Africa, to establish limits of applicability and to determine the accuracy of prediction. For these reasons a proper investigation was excluded, as time was limited, and it was therefore decided to do a thorough literature survey to extract typical characteristic curves for representative pavement materials and to draw a diagram depicting the generalized characteristic relationships between suction and moisture content for these soils.

The use of these curves in conjunction with the equilibrium suction values will give a first estimate of a design moisture content and will also serve as a check on the prediction value resulting from the empirical techniques. It is emphasized that this is a pilot study and heavy reliance is placed on experience from abroad.

8.2 Background

For many years now the British and specifically the Australians (Aitchison and Richards, 1965) have attempted to test the subgrade material in a representative unsaturated state. Various techniques have been developed which are described elsewhere (Haupt, 1978). Some of these techniques are based on the observation that there is a relationship between climate and the equilibrium suction under sealed pavements. As pointed out before, highly refined empirical models for moisture prediction are available for local conditions, but it was decided to develop a rational technique based on suction in an attempt to increase the prediction accuracy.

Figure 8.1 shows the data and relationships developed for conditions in Australia.

A few observations may be made from Figure 8.1:

(a) The observed points are in general agreement with the mean line.

(b) Serious discrepancies between laboratory and gypsum block readings are apparent.

(c) The measured matric suction usually lies below the design line (i.e. more weight is given to total suction measurements).

(d) Appreciable amounts of soluble salts are present in most of the samples.

The main reasons why this relationship could not be used directly for local conditions are:

(a) no readily available local information is available to support it;

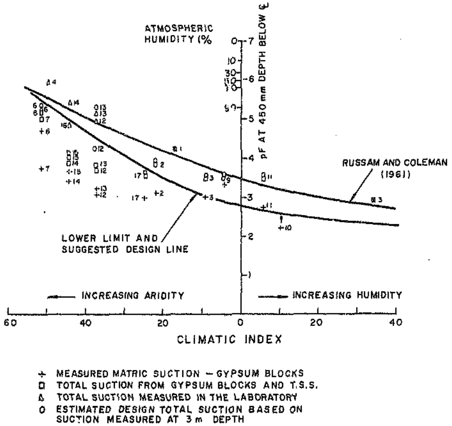


FIGURE 8.1

DATA FROM ROAD SITE INSTALLATIONS AND POSTULATED DESIGN CURVES FOR VALUES OF SUBGRADE SUCTION

(b) matric suction is the main component affecting strength in a sample, and the osmotic component can only be incorporated in a membrane-like situation; and

(c) local subgrade soils generally do not contain appreciable amounts of soluble salts.

For these reasons it was decided to collect as much local information on suction under covered areas as possible, and to establish a relationship applicable to local conditions.

8.3 Relationship between suction and climate in southern Africa

8.3.1 Climatic parameters

It was decided to relate suction to more than one climatic parameter so that the best relation could be selected. Those parameters had to be chosen which would be readily available to prospective users and so mean annual rainfall (MAR), Thornthwaite's I-index (I) and Weinert's N-index (N) were chosen. Figure 8.2 shows the mean annual rainfall contour map for South Africa, Figure 8.3 the Weinert N-index contour map and Figure 8.4 the Thornthwaite's I-index contour map.

The mean annual rainfall map was compiled from data for the 30 years period from 1921 to 1950.

The N-index contour map is available from NITRR and is based on a climatic index defined by Weinert (1974).

The I-index contour map was compiled from data given by Schulze (1958; 1947).

8.3.2 Available local information

Prior to this investigation, only the NBRI had reported any work done on in-situ suction measurements in southern Africa. De Bruijn (1963, 1965, 1965a, 1973, 1973a, 1975) carried out the majority of work done locally between 1963 and 1975, when he measured suctions and suction changes under various types of covering, e.g. plastic, sand, etc., using mainly gypsum-blocks. Simons and Williams (1963) also reported on moisture studies they carried out near Pretoria. All these reports, together with all the suction measurements under covered areas ever taken by NBRI, were carefully sieved in order to build up a data base. All measurements taken under pavements or under plastic or rubber coverings were used. Although many more sites were measured, only 17 sites, scattered throughout South Africa and South West Africa-Namibia, were measured regularly and under the right type of covering. Many sites were located close to Pretoria, but the average of these values was used here.

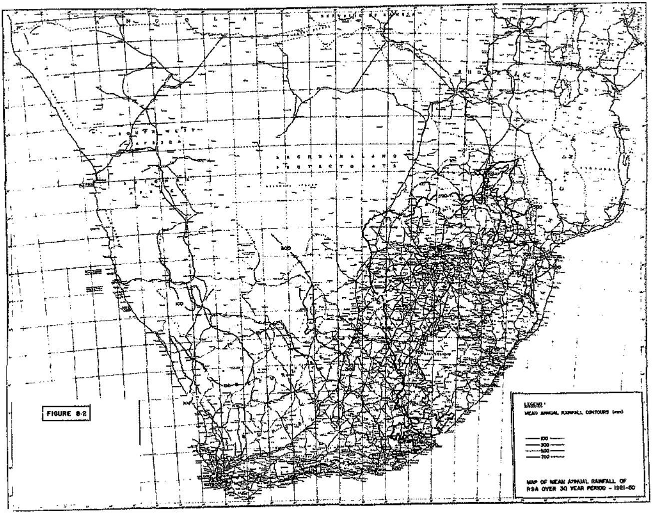
Five sites were also instrumented by NITRR with psychrometers, viz. three near Pretoria, one near Storms River in the Cape and one near Keetmanshoop in S W A. The three Pretoria sites were included in the NBRI sites for Pretoria, which means that effectively there was reliable information available on 19 different locations in southern Africa (cf. 17 sites for Australia).

It must be noted at this juncture that all local suction measurements are total suction measurements because they were done with gypsum blocks and psychrometers. At none of the sites instrumented were there any obvious signs of salts present, at least not in its crystallized form. It is therefore reasonable to assume that less than 0,2 per cent (by weight) salts were present in any one sample (Netterberg, et al, 1974). Although moisture content measurements were not available for all samples, it is reasonable to assume that all the samples used in areas wetter than pF 3,5 had moisture contents in excess of at least 10 per cent. With 10 per cent moisture and 0,2 per cent salt in the soil, the salt concentration in the soil water is 2 per cent. The molality of the solution is 0,34 which causes an osmotic potential of ± 1 500 kPa (pF 4,2). This is a higher suction than the total suction measured in these regions, which is indicative of less salts and higher moisture contents. The fact that the samples used have shown good correlation with matric suction results from abroad, makes it reasonable to assume that negligible amounts of salts were present in most samples, in which case the term "total suction" on each graph is interchangeable with "matric suction". The results of salt content measurements and the associated osmotic suction components for the samples, where this could be determined, are summarized in Table 8.1.

No written records of any work done at NBRI on salt contents could be found, but De Bruijn reported (15/2/80) that he had carried out tests on his sites and that less than 0,1 per cent salts had been present in all cases. He also said that the percentage soluble salts had been so low that they had <u>no</u> influence on the gypsum block readings.

It is for these reasons that it was decided that matric suction could be used in this investivation instead of total suction.

Table 8.2 summarizes the climatic and suction information for the 19 sites investigated.


SOUTH WEST AFRICA
BECHUANALAND PROTECTORATE
FIGURE 8-2
LEGEND:
MEAN ANNUAL RAINFALL CONTOURS (mm)
100
300
500
700
MAP OF MEAN ANNUAL RAINFALL OF RSA OVER 30 YEAR PERIOD - 1921-60

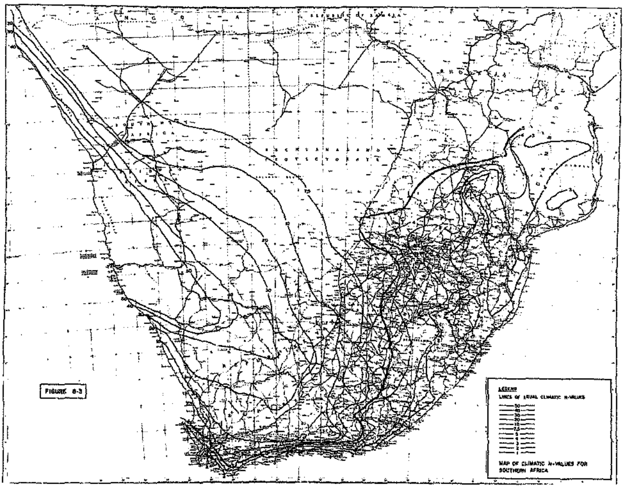


FIGURE 6-3
LEGEND
LINES OF EQUAL CLIMATIC N-VALUES
50
40
30
20
10
7.5
5
4
3
2
1
MAP OF CLIMATIC N-VALUES FOR SOUTHERN AFRICA

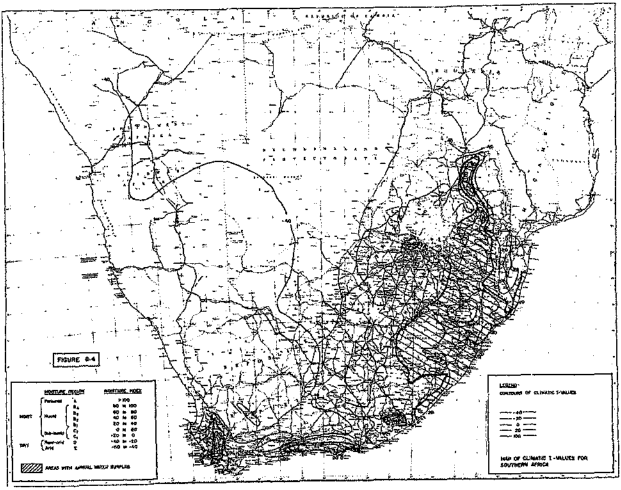

LEGEND
CONTOURS OF CLIMATIC I-VALUES
-40
-20
0
20
100
MAP OF CLIMATIC I-VALUES FOR
SOUTHERN AFRICA
MOISTURE REGION
MOISTURE INDEX
AREAS WITH ANNUAL WATER SURPLUS
FIGURE 8-4

TABLE 8.1: OSMOTIC SUCTION FOR TESTED SUBGRADE SOILS

Location	Keetmanshoop	Storms River	Pretoria
Salt type	Chloride and traces of carbonate	Chloride	Chloride
Conductivity ($mScm^{-1}$) at 25 °C	3,1	1,0	1,0
Soluble salts (%)	0,12*	0,02	0,02
Moisture content	5	30	20
Molality	0,41	0,01	0,02
Osmotic suction (kPa)	2300	10	40
Osmotic suction (pF)	4,36	2,00	2,68

*Reduced by 50 per cent because (a) the sample was prepared wet and the water boiled off so that no salts were lost. All the salts were thus washed off the coarse aggregate into the fine fraction (±25 per cent over-estimate), and (b) the tests were done on the 0,425 mm fraction and not the 6,7 mm, which also causes an over-estimate of 25 per cent.

From the literature it is clear that the erection of buildings and roads does not affect the soil moisture regime significantly in very humid regions (Magoebaskloof) and very arid regions (Mariental) (De Bruijn, 1973a). In more moderate climates (Onderstepoort and Vereeniging), there appears to be a marked increase in moisture content (decrease in suction) when the soil is covered. Although the soil under the covered areas is generally somewhat wetter than that in the open veld, it is not subject to large changes. Changes do, however, occur and for this reason it was decided to use the term "characteristic suction" instead of "equilibrium suction". The characteristic suction of a location means a suction value such that there is only a specified chance that the actual suction (in pF) at that location will be less than the characteristic value. For this investigation, insufficient data were available on all holes to do a proper statistical analysis and to define a reliable confidence interval for the characteristic suction, but Table 8.3 shows the sites where sufficient information was available to define the 85 per cent confidence interval. (This means that there is only a 15 per cent chance that the actual suction will be less than the characteristic suction.)

TABLE 8.2: CHARACTERISTIC SUCTION VALUES WITH CORRESPONDING CLIMATIC VALUES I, N AND MAR FOR EACH LOCATION USED

No.	Location		Total suction pF	Estimated matric suction pF	Climate		
					N Weinert	MAR mm	I Thornthwaite
1	Keetmanshoop (S.W.A.)	X	4,6	4,3	48,2	125	-50
2	Mariental (S.W.A.)	X	3,7	3,7	33,0	190	-48
3	Britstown	X	3,7	3,7	19,0	260	-42
4	Richmond (Cape)	+	3,6	3,6	13,5	322	-33
5	Welkom	X	3,2	3,2	4,7	570	-15
6	Onderstepoort	+	3,3	3,3	2,4	710	- 9
7	Vereeniging	X	3,0	3,0	2,4	670	- 9
8	Bapsfontein	X	3,0	3,0	2,4	750	- 5
9	Pretoria	X	3,0	2,8	2,4	750	- 4
10	CSIR	X	3,0	2,8	2,4	750	- 3
11	Barkly East	+	2,9	2,9	3,9	622	- 3
12	Johannesburg	+	3,2	3,2	2,3	750	3
13	Tzaneen	+	3,6	3,6	1,3	1000	15
14	George	X	2,0	-	1,2	1040	50
15	Magoebaskloof	+	2,0	2,0	1,0	1250	75
16	Rondebosch (Cape)	+	3,0	3,0	3,7	600	-10
17	St. Helena (O.F.S.)	X	3,0	3,0	4,7	570	-15
18	Freddie South (O.F.S.)	X	3,0	3,0	4,7	570	-15
19	Bloemfontein	X	3,7	3,7	5,3	560	-14

X Values underneath pavement

\+ Values underneath covered areas

The nature of the data makes it very difficult to select exactly the 85 per cent confidence interval value for suction. Table 8.3 shows that for 9 of the 19 sites, the tabulated values for suction in Table 8.2 lie close to the 85 per cent confidence interval. For the other 10 sites, insufficient information is available to give actual values, but it is resonable to assume that they will be very similar in all respects.

TABLE 8.3: THE CONFIDENCE INTERVALS OF SUCTION READINGS FOR SOME LOCAL SITES

Location	Matric suction (pF)	Confidence interval (%)
Keetmanshoop	4,3	90
Magoebaskloof	2,0	90
Pretoria	2,8	90
Bloemfontein	3,7	85
Tzaneen	3,6	85
Vereeniging	3,0	83
Mariental	3,7	83
Richmond	3,6	82
Barkley East	2,9	80

The data in Table 8.2 were then used to establish the best relationships between suction and the different climatic parameters. As pointed out before, total suction measurements were made, but seeing that there is not sufficient evidence of appreciable amounts of salt present, this will be the same as the matric suction.

Figure 8.5 shows the relationship between rainfall and the characteristic matric suction for southern African conditions. It is clear that there is general agreement between rainfall and suction, the correlation coefficient between the two parameters being 0,79. The best fit exponential equation is:

$$pF = 4{,}52e^{-5{,}78 \times 10^{-3}\cdot(MAR)}$$

Figure 8.6 shows the relationship between Weinert's climatic N-index and suction. The correlation coefficient between the two parameters is 0,78 and the best fit logarithmic equation is:

$$pF = 2{,}56 + 0{,}99 \log_{10}(N)$$

Figure 8.7 may be compared with Figure 8.1. Figure 8.1 gives the data for Australian conditions, while Figure 8.7 gives those for local conditions. Curve 1 on Figure 8.7 shows the best exponential curve that could be fitted to the data with a correlation coefficient of -0,84. Curve 2 shows the proposed local curve, which was developed by giving more weight to the more reliable local data, i.e. the points where both the matric and osmotic suction components were known and where the measurements were taken under the road.

Table 8.4 summarises the correlations between suction and the various climatic parameters. It is clear that the climatic parameter which correlates best with suction is the Thornthwaite index.

TABLE 8.4: CORRELLATIONS BETWEEN CLIMATE AND SUCTION

	Curve fitting			
	Exponential	Log	Power	ln
MAR	0,79	0,36	0,75	0,79
N	0,65	0,78	0,74	-0,73
I	0,88	0,88	0,73	0,84

8.3.3 Local suction contour map

Using curve 2 in Figure 8.7, a suction contour map of the RSA could be compiled. Figure 8.8 shows this map, which can be used to determine the characteristic matric suction at a depth of 0,3 to 1,0 m below the surface of a covered area anywhere in the RSA. There is approximately a 15 per cent chance that the suction (pF) is lower than the characteristic value and an 85 per cent chance that it is higher and the material thus drier.

8.4 The characteristic relationship between suction and moisture content for local representative subgrade materials

The approach adopted to establish this relationship was identical to that used to develop a relationship between climate and suction. An

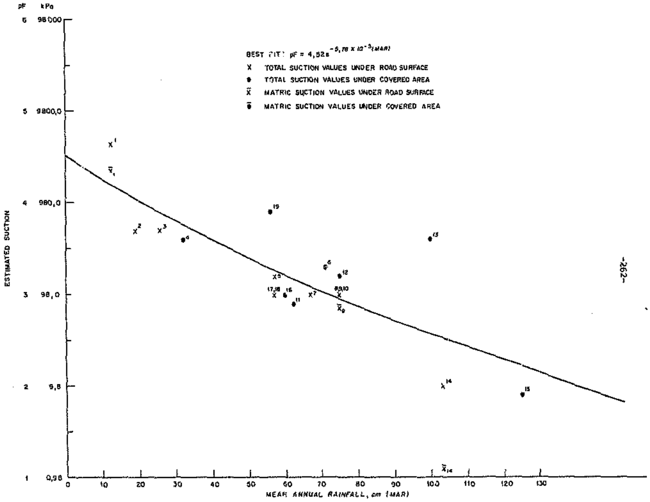


FIGURE 8.5

RELATION BETWEEN CLIMATE AND SUCTION FOR SOUTHERN AFRICA

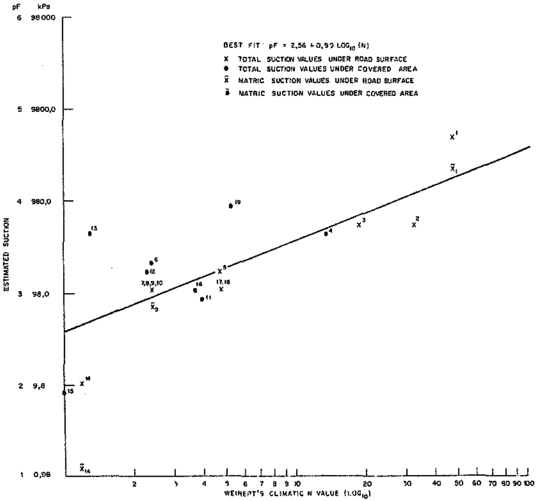


FIGURE 8.6

RELATION BETWEEN CLIMATE AND SUCTION FOR SOUTHERN AFRICA

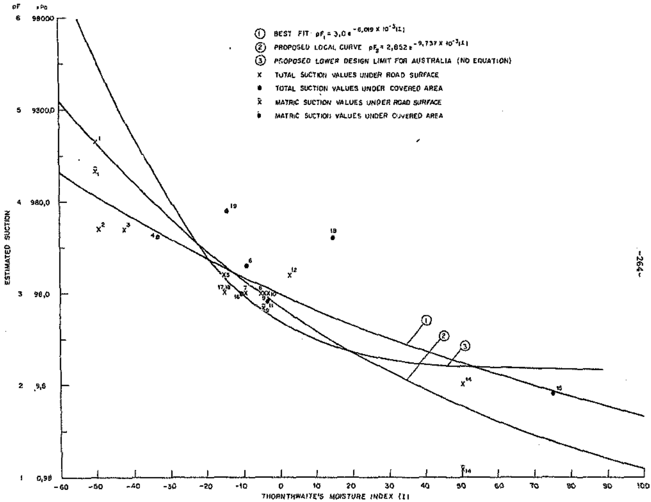
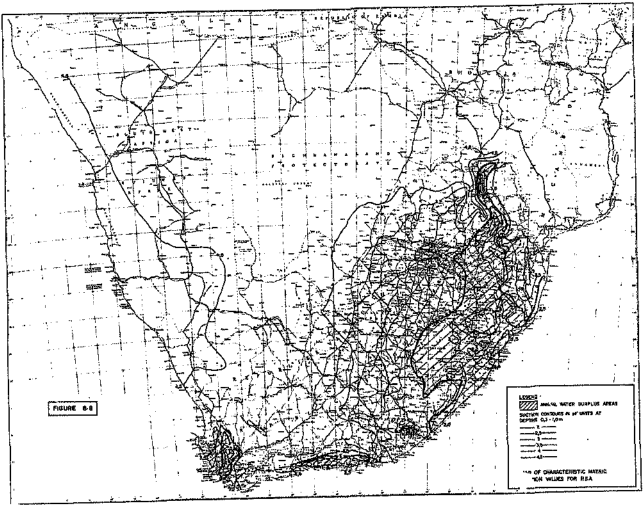


FIGURE 6.7

RELATION BETWEEN CLIMATE AND SUCTION FOR SOUTHERN AFRICA



LEGEND
ANNUAL WATER SURPLUS AREAS
SUCTION CONTOURS IN pF UNITS AT DEPTHS 0,3 - 1,0m
2
2,5
3
3,5
4
4,5
OF CHARACTERISTIC MATRIC
VALUES FOR RSA
ANGOLA
SOUTH WEST AFRICA
BECHUANALAND PROTECTORATE

FIGURE 6-8

extensive literature survey was carried out in order to compile a data base of published characteristic curves. Figure 8.9 results from this literature survey. Local information that was included in the compilation of Figure 8.9 com_ r ses mostly work done by De Bruijn (1963, 1973, 1975) and Brackley (1976). Information from abroad formed the basis for Figure 8.9 and the bulk of the work was done by Black (1962), Croney, Coleman and Russam (19?3), Croney and Coleman (1948, 1954), Croney, Coleman and Bridge (1952), [illegible] et al (1972) abd Livneh and Ishai (1975).

The information retrieved from the literature resulted in the dotted lines in Figure 8.9. A distinction between sand, silt and clay is made. These curves were slightly adjusted after a laboratory investigation into the suction-moisture content relationship of some local subgrade soils. The parameter "adjusted liquid limit" was used to divide the range, because this parameter gave much better correlations with material properties than the simple liquid limit. The solid lines represent the areas verified for local soils. This is a very general diagram, but it may be very useful in providing a first estimate of the moisture content to be expected under the pavement.

It should be noted that all suction measurements taken for this investigation were done with psychrometers. This means that total suction was measured, but, as pointed out before, in this case it may be equated to the matric suction because of the small amount of soluble salts present.

Figure 8.10 shows a simplified diagram relating matric suction to moisture content for different local soils.

8.5 Proposed method to determine a characteristic moisture content for design

The following procedure should be followed to arrive at a first estimate of the eventual moisture content under a sealed surface:

(a) Find the location of the site on Figure 8.8 and interpolate the characteristic matric suction for that site.

(b) Determine the liquid limit and grading properties of the soil present on the site.

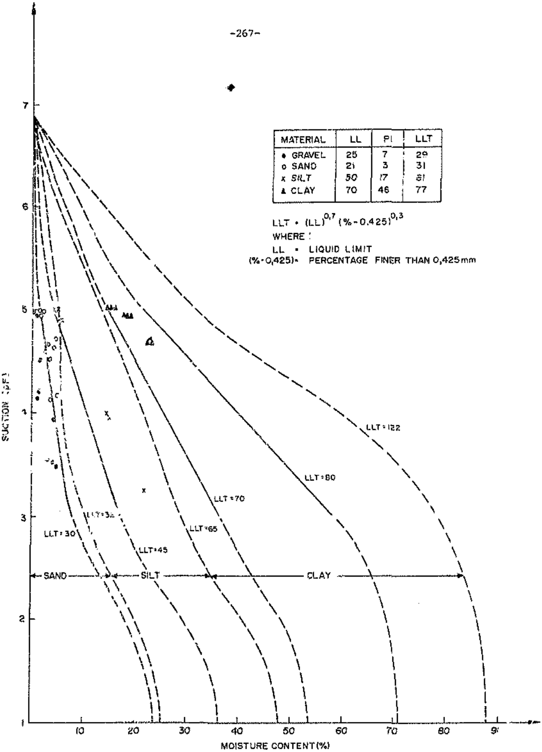


FIGURE 8.9

GENERALIZED CHARACTERISTIC RELATIONSHIPS BETWEEN SUCTION AND MOISTURE CONTENT AS TAKEN FROM LITERATURE AND ADJUSTED FOR LOCAL TESTS

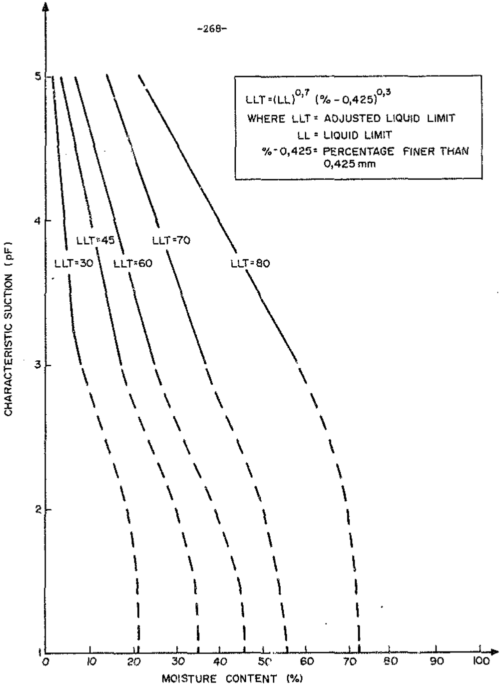


FIGURE 8.10

GENERALIZED RELATIONSHIP BETWEEN CHARACTERISTIC SUCTION AND MOISTURE CONTENT

940-4-2779/1 KT

(c) With the known suction and adjusted liquid limit, Figure 8.10 may be used to read off the relevant moisture content.

8.6 Conclusions

The following conclusions may be drawn from this chapter:

(a) In order to improve on the prediction accuracies attainable with the empirical models, a preliminary investigation into a rational prediction technique for local conditions was carried out.

(b) This constitutes a preliminary study only and heavy reliance is placed on experience from abroad.

(c) Although measurements of the in-situ moisture content and the percentage soluble salts present in the material tested were not available in all cases, indications are that negligible amounts of salts were present, which means that the osmotic component would be very small. More work is required in this regard.

(d) Because of the uncertainty as to whether equilibrium conditions do in fact occur, and because of the many factors influencing the suction, the term characteristic suction (matric potential) is defined so that the statistical scatter around the mean value may be incorporated for design purposes. The characteristic suction is defined as that suction value such that there is only a specified chance that the actual suction (in pF) will be less than the characteristic value.

(e) It was found that Thornthwaite's climatic index correlates better with the suction than any of the other climatic parameters tried for local conditions.

(f) Using the relationship between suction and Thornthwaite's I-index, a characteristic suction contour map could be drawn for local conditions.

(g) After a limited laboratory study and a comprehensive literature survey, curves showing the generalized relationship between suction and moisture content for different soils could be drawn up.

(h) This chapter provides a rational method for determining a characteristic moisture content for design purposes. It is suggested that it be used in conjunction with the empirical methods developed in chapter 7 to serve as a check.

CHAPTER 9

PROPOSALS FOR PAVEMENT DESIGN

CHAPTER 9

PROPOSALS FOR PAVEMENT DESIGN

CONTENTS

CHAPTER 9

PROPOSALS FOR PAVEMENT DESIGN

9.1 Introduction

In the previous 2 chapters empirical and provisional rational techniques for the prediction of an equilibrium moisture content were developed. It is the aim of this chapter to show how these techniques, in particular the empirical meth[illegible] used in pavement design. Although it was not one of the ma[illegible]is investigation to show how the design CBR will be affected by a [illegible]ge of a design moisture content, this note will report on a preliminary study carried out to show how the standard soaked CBR may be adjusted to provide for unsaturated in-service conditions.

This chapter is logically divided into 3 categories. Firstly, the development of the CBR pavement design procedure will be summarized to highlight the initial aims and their change with time. Secondly, some background information will be presented on the design procedures in other countries at present and how these relate to the practice in South Africa. Finally, after the need for modifications to the local procedure has been motivated, a method is proposed whereby moisture may be rationally incorporated in the design procedure. Guidelines will also be given as to how the soaked CBR and the NITRR method of predicting heave may be adjusted for unsaturated conditions.

9.2 The development of the CBR pavement design method

In designing flexible pavements, the strength of the subgrade is a principle factor determining the thickness of the pavement, as the pavement must be designed to withstand successive applications of anticipated wheel loads without undue distress to any layer, including the subgrade. It should be clear that it may be very rewarding to devote a fair amount of effort to establish the true strength properties of the subgrade, as large financial savings could be effected. A design procedure incorporating this strength will lead to more economical designs. Elsewhere (Otte, 1979), more detailed information about the development of the CBR test and design procedures is available, but it will serve a useful purpose here to very briefly recall the highlights of the historical development.

The test was devised after an investigation by the California Roads Department into failed pavements during 1928 - 1929 (Porter, 1942). It was first used as a classification test to differentiate between high and low quality crushed rock base material and particularly to isolate those materials likely to lose strength after prolonged soaking in water.

The use of the CBR test in California was extended in the late 1930s (Porter, 1938) to the evaluation of subgrades. Investigations were made involving a number of roads in California, representing various stages of deterioration, and a relationship was developed in this way between the soaked CBR of the subgrade and the thickness of construction regarded as "adequate" for existing traffic conditions. This relationship is shown in curve "B" on Figure 9.1. To take account of the growth of commercial traffic, a second curve "A" was developed to represent the requirements for future "average traffic conditions" (Grumm, 1942). Traffic was not quantified more precisely than this and the question of design life was not considered.

The fact that the original Californian design curves were related to the soaked CBR value is sometimes taken as meaning that the curves take into account the weakest conditions which could arise in the subgrade. This is clearly <u>not</u> so. The roads investigated in developing the design curves presumably embraced a wide variety of soil drainage conditons and the designs must therefore have referred to the average soil moisture conditions likely to have occurred in practice in Californa in the 1930s. They were arrived at by establishing a purely empirical relationship between the test values and the suitability of base and subgrade materials after an extensive investigation of pavement failures and testing the materials. Not everybody agrees with this, arguing that Porter had in fact used the CBR test as a subgrade shear strength evaluation test and based his design curves on this, thus catering for the worst possible condition.

This is not strictly correct, because although the penetration test provides a measure of the cohesion and internal friction and thus of the

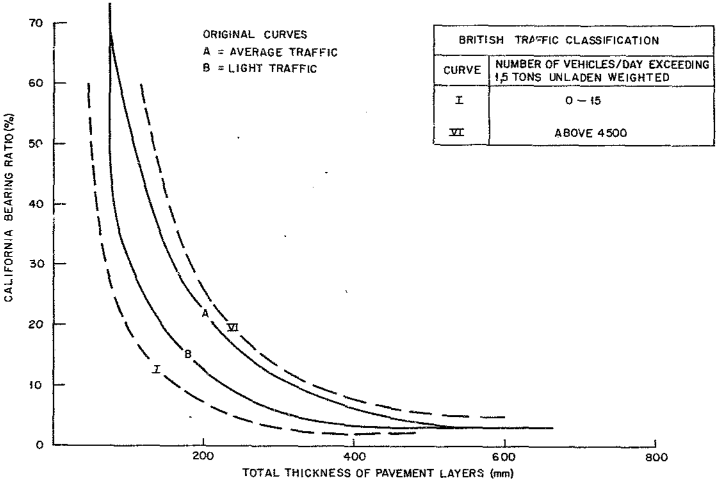


FIGURE 9.1

COMPARISON BETWEEN ORIGINAL CBR DESIGN CURVES AND BRITISH CURVES shear strength of a material, and although Porter realized this, it was not incorporated in the original design curves. The measure of shear strength of the tested material in the CBR test was not originally related to the actual shear stress levels attained or developed in the field. The CBR value arrived at from the test was simply empirically related to a specific thickness of pavement, which was observed during its service life to be either suitable or unsuitable. This is borne out by the fact that Porter did not find this method excessively over-conservative in California, because average moisture conditions in California had been considered. Had the CBR test been a true test for shear strength and used as such in the design procedure, it would mean that the material was tested under soaked conditions and the design procedure would lead to over-conservative results wherever the in-situ moisture conditions were drier than saturated, because in such places the shear strength would be higher than that measured.

Porter (1949) stated that the sample was soaked to "reach the adverse state of moisture which is usually present in the subgrade under normal drainage and climatic conditions". There are no reports available to substantiate generalizations like this, and it is presumably based on Porter's personal experience of moisture conditions in pavement layers in California at that time. When considering the present day drainage provisions and general construction procedures, and bearing South African experience in mind, it is inconceivable that more than 50 per cent of all Californian subgrades ever reach saturation at present. It should, however, be clear that the intention right from the start with this test was for it to be carried out at conditions representative of in-situ field conditions, as Porter (1949) stated "the soil often reaches the state of wetness and compaction comparable to the soaked specimen in the bearing test".

Nevertheless, during World War II the U.S. Corps of Engineers adopted the CBR design method for design of airfield pavements after an evaluation of other possible methods. To enable the design curves to be extrapolated to cater for aircraft wheel loads, it was necessary first to ascribe to the original Californian curves maximum wheel loads. For the commercial vehicles then in use, a maximum wheel load of 53 kN was ascribed to curve A in Figure 9.1. This value was taken from wheel load measurements made by the Division of Highways, California, and the reasoning of the U.S. Corps of Engineers was that the Californian designs h[illegible] proved satisfactory for a limited number of these wheel loads, comparable to the frequency of aircraft loading.

To extrapolate the design curves from these modest wheel loads to 660 kN relevant to aircraft loading, the Boussinesq equations for circular loading were used. Subgrade stress lev[illegible]ls under the 53 kN wheel were calculated and assumed "safe" and the cover required to ensure these "safe" stresses under heavier loads was calculated.

No provision for layered systems material properties were made at this stage, because the Boussinesq analysis showed that the vertical stress distributions are independent of material properties. The material properties (CBR) were still only incorporated empirically in the design procedure, through the adoption of curve A as a design basis. It is thus clear that although a rational approach had been used to extend the design curves to other wheel loads, it was still related to average moisture conditions in Californian pavements in the early 1930s. It is, however, probably reasonable to assume that these moisture conditions were close to saturation due to inferior construction techniques, insufficient drainage provision, and less stringent geometric standards prevailing at that time. This means that the CBR test was in fact done at conditions thought to be representative of field conditions. Full scale field tests confirmed that these extrapolations were correct (Porter, 1949a; Hansen, 1949).

The principle of an equivalent single wheel load, number of repetitions and design life were also incorporated in the design procedure during later years.

To summarize, it can be said that although the CBR test was initially used in California as a classification test, it was later used as an indication of shear strength in the subgrade. If it is viewed as a strength test, then it should be carried out at conditions representing field conditions. Porter persisted in testing at the soaked condition, because that was presumably the moisture state in pavements in 1929 in California. The cover curves developed for this test were, however, based on a wide variety of roads and moisture conditions in Calfironia.

This means that if one uses this design method, one can expect, as experience in California has shown, that the resulting pavement will give good service, provided the in-service moisture conditions do not become worse than those in California in the early 1930s. Seeing that these conditions were close to saturation, the chances are virtually zero, except for isolated spots of bad drainage, of this happening in South Africa. It is for this reason that it is strongly recommended in this dissertation that the CBR for pavement design purposes should be representative of field mositure conditions.

9.3 Variations of the general pavement design procedure

9.3.1 The British method

In 1946 the British decided to adopt the CBR test as a basis for the design of flexible pavements, but unlike California, they immediately employed the fact that the test provided an assessment of the strength likely to be achieved in the subgrade after construction. Because of uncertaint[illegible] relating to the exact test procedure and to the method used to ascribe [illegible] loads to the original curves, it was decided to prepare new design curves linked to CBR tests carried out at the equilibrium moisture content and dry density conditions expected under the completed road pavement. Another major difference in approach was that the British considered the number of expected vehicles per day exceeding 1,5 tons unloaded weight as the load applied to the pavement, instead of considering a specific wheel load. Later they introduced the idea of standard axles. Their curves were developed partly from failure investigations similar to those carried out in California and partly from evidience gained from full-scale experimental pavements (Croney, 1977).

Figure 9.1 also shows the British curves plotted on the same scale as the original Grumm curves and it is clear that they are very nearly identical, in that only the extreme curves I and IV are drawn and all the intermediate curves will cover the original envelope entirely.

Because the British wanted to represent field conditions rather than soaked conditions, they carried out extensive research (Croney et al, 1952; Woodman et al, 1963; Russam and Coleman, 1961; Thorntnwaite, 194[illegible], Croney et al, 1958; Russam, 1962) to establish methods of determining the field equilibrium moist[illegible] content and the influence of moisture content and the density on the CBR. For British conditions the equilibrium suction of a specific site can either be calculated or estimated from past experience. Samples of the relevant material may now be tested for moisture content at various suctions in order to establish the moisture content-suction relationship for that specific soil. From this information, an estimate of the moisture content at which the CBR should be tested can be made. This type of testing requires a well-equipped laboratory, which may not be available, and Road Note 29 (RRL, 1970) prescribes the use of a table listing estimated laboratory CBR values for British soils compacted at optimum moisture content and tested at the natural moisture content which "for most purposes" will give appropriate CBR values.

It is also specified that where specialist equipment and experience is available, the CBR test may be carried out on recompacted samples in accordance with BS 1377:1967. The moisture content and density conditions used in the test should reproduce as closely as possible the conditions likely to apply under the road after construction. In-situ CBR measurements are also allowed for cohesive soils.

It is clear that the British, in a climate that is generally far wetter than the southern African climate, allow certain modifications to the CBR test procedure in order to take unsaturated conditions into account.

9.3.2 British method in tropical and subtropical areas

After extensive research into moisture conditions in tropical and subtropical climates, the Road Research Laboratory produced Road Note 31 (1977) in which a method is presented by which the design CBR can be determined at the relevant field moisture content.

They (O'Reilly et al, 1968) found that in the tropics, the subgrade moisture conditions under impermeable road pavements can be classified into three main categories, as explained in Chapter 3. This chapter also deals with the procedure of estimating the moisture content in each of the above categories.

It is important to ensure good drainage and an impermeable pavement when these moisture contents are used. It is also practice to leave samples of cohesive soils in plastic bags for 24 hours so that the excess pore water pressure can be dissipated before penetration.

The recommended procedure for determining the design CBR is to compact the soil at several moisture contents (not OMC) and at least two levels of compaction, measuring the CBR on each sample. The design CBR can be interpolated from this and the consequences of errors evaluated, since both the influence of density and moisture content will be known.

9.3.3 Australian methods

The Australians have found, after extensive research into moisture conditions in their country, that there is no single method of predicting the equilibrium moisture content available. They prescribe different methods of determining the equilibrium moisture content and these are summarized in Chapter 3. The method used will depend on the data and equipment available. What is of importance here is to note that they have deviated from using the soaked CBR as a design standard, because research and experience have shown it to be over-conservative.

Some Australian researchers (Loxton et al, 1952) have also promoted the idea of compacting the sample initially at its ultimate moisture content and then testing it immediately. This has also been suggested in Britain, but the Corps of Engineers (Turnbull and McRae, 1952) have serious doubts about it. They have found that the stress-strain characteristics of a compacted soil are to a great extent a function of moulding water content on both soaked and unsoaked samples. This means that the stress path the sample follows in the laboratory must represent the stress path experienced in the field. The writer feels that this is a valid point and it is not suggested that this procedure be followed.

9.3.4 The New Zealand method

Very little research on moisture conditions in New Zealand has been carried out and they stick to the soaked CBR test as a basis for designing flexible pavements, provided the subgrade is weak (CBR < 5) (Dunlop, 1977;

McDowell, 1972). When the subgrade CBR is higher than 5, the sub-base design is based on New Zealand experience resulting in pavements considerably thinner that would result by using the original CBR curves. If the CBR of the subgrade is less than 5, the subgrade is strengthened by some method of stabilization for 150 mm and the rest of the pavement is then designed utilizing this stronger subgrade strength. Thus, although the New Zealanders have not changed the moisture conditions of testing, they have departed from the original design procedure.

9.3.5 The U.S.A. method

In most states of the U.S.A. the soaked CBR is used as a basis for flexible pavement design. Leading researchers in the pavement design field (Monismith, 1979; Witczack, 1979) state, however, that the CBR test must be considered as a shear strength test and that it must be done at anticipated field conditions. The Corps en Engineers allow a rule of thumb reduction in required thickness of 20 per cent where the rainfall is less than 380 mm and the water table deeper than 5 m. It is accepted practice in the U.S.A. that if the field moisture content is expected to be below the soaked value, the CBR is tested at a moisture content lower than OMC.

It should be explained here that the CBR is not used directly in most design procedures in use in the U.S.A. The AASHTO design procedure, for instance, introduced an arbitrary soil support value which cannot be directly obtained by testing. Each design agency using this guide must establish correlations between standard soil tests (e.g. CBR, R, Triaxial strength) and the soil support value. Many agencies (Utah State Department of Highways, 1966, 1967; Van Til et al, 1972) have produced such correlations and they are available in literature. In order to make the procedure more applicable to other environmental regions, a regional factor (R) was introduced together with soil support value. The procedure adopted in South Africa in 1972 (TRH 4) (and still in official use) assumed a CBR value of 3 and made no provision for other subgrade conditions, althougn it incorporated the R factor to make provision for the environmental influence.

9.3.6 The Israeli method

The Israelis have largely been led by research and practice in other countries and they apparently favour the British method of determining the moisture content at which the test should be carried out.

Until the early 1960s, the Israelis selected soaked CBR values in the vast majority of cases (Kassif et al, 1969), but after work done in Britain and Australia they realised that this may lead to over-design (Livneh and Ishai, 1975) and in recent times they tend to test in unsaturated conditions.

9.3.7 The Canadian method

After work done by McLeod (1947) the Canadian Department of Transport based its pavement design procedure not on the soaked value but rather on the in-situ CBR values of existing roads under similar climatic conditions. The method was based on plate bearing results and this is still most commonly used to characterize the subgrade strength, but all strength tests are carried out in-situ.

9.3.8 Economic Commission for Africa

Many of the roads built in African countries are built by this commission, using mainly British and French experience. They have stated at a recent conference (Parker et al, 1974) that there is an increasing tendency in hot climates to determine the maximum moisture content that is likely to develop in the subgrade and to design the thickness of the pavement on the basis of the strength of the subgrade at this moisture content. In most cases they have found the relevant moisture content at which testing should take place to be the optimum moisture content at BS light or Std. AASHTO.

9.3.9 The South African method

Regardless of climate or actual field conditions the South African authorities insist on soaked conditions for testing. This approach may have justification where no accurate information about actual moisture conditions is available. This is no longer the case and the main reasons for proposing a change in the traditional approach may be summarized as follows:

(a) As a result of recent work done in South Africa and reported on in this dissertation a mass of information is available about moisture conditions locally. Highly reliable prediction models are available and, as it is clear that saturation seldom is reached in the design life of most pavements in certain climatic areas, these well-proven models should be incorporated in the design procedure.

(b) The CBR test was considered from the start, in the early 1930s, as a test to be carried out under "the adverse state of moisture which is usually present in the subgrade under normal drainage and climatic conditions" (Porter, 1949). It was said that the sample had to be prepared in such a way as to "truly represent the condition of (a) consolication, (b) moisture content and (c) the structure" that pertains to the field. As one of the main advantages and favourable features of the test was quoted its flexibility, "that it can be conducted on a laboratory specimen in which the moisture content can be adjusted to a given design value" (Turnbull, 1949). One of the main reasons for accepting saturation was given as "as information on moisture conditions under pavements is very conflicting, the Corps of Engineers had no other course than to assume the maximum condition to ensure conservative design" (Turnbull, 1949). It is very interesting to note that it was categorically stated during the 1949 symposium of the CBR design method that "the criticism (of over-conservative design) is believed to be valid" in cases where saturation is never reached under a pavement.

(c) Most leading design agencies have made some provision for the fact that the pavement layers (especially the subgrade) never reach saturation under certain conditions. The exact measures taken have been summarized in this dissertation. Most of these provisions have resulted from limited moisture investigations and the performance of pavements in different environments. Extensive moisture studies in South Africa provide a good basis for modifying the interpretation of the CBR test results.

(d) Heavy vehicle simulator tests carried out in South Africa on light pavements (Maree, 1979) particularly indicate that they perform much better than one can theoretically predict and it is believed by the author that this may mainly be ascribed to the fact that the pavement layers hardly ever become saturated. In spite of the highly reliable moisture prediction techniques available, this is an even more persuasive argument because it provides proof that many of our roads are in fact over-designed.

(e) Although the CBR design method was developed to result in "permanent construction" involving "only minor maintenance" (Turnbull, 1949), this philosophy is not always necessarily the best for all types of pavement. One might be willing to accept more isolated failures on, say, lightly trafficked roads, and adapt one's maintenance program accordingly. Although this represents a major departure from traditional precedures, it places us in line with the rest of the world and it will in any case initially only affect lightly trafficked roads until more knowledge becomes available from performance data. It must also be borne in mind that all pavement design procedures are based on field data with a certain scatter associated with it. This means that even if tests are carried out on unsaturated samples, the resulting thickness will probably still lie within the general band of accepted designs.

9.4 The incorporation of moisture in local pavement design

From the foregoing it should be clear that there exists overwhelming evidence supporting the incorporation of moisture in some way in pavement design. This paragraph will suggest a method whereby the empirical models developed in Chapter 7 may be incorporated not only in pavement design but also in various geotechnical problems like the prediction of collapse and heave.

9.4.1 Estimating a design moisture content

The various equations summarized in Table 7.4 and the associated accuracies clearly provide a method for predicting a design moisture content. It is recommended that a characteristic moisture content be used for design purposes (similar to the characteristic strengths and loads used in structural design), because such an approach would be valid whether equilibrium moisture conditions are established or not. The characteristic moisture content (minimum or maximum) may be defined as that moisture content below or above which only a certain percentage of actual moisture content values will fall respectively.

When applying the formulae in Table 7.4, the average (equilibrium) moisture content for that material in that climate will be predicted. If one is interested in the maximum moisture content likely to occur, the recommended procedure makes provision for increasing the average moisture content by a certain amount, depending on the prediction accuracy of the model used, in order to limit the chance that the increased moisture content will be exceeded, to acceptable limits. First one has to decide on what chance one is prepared to risk that the actual moisture content will exceed the predicted one. In structural engineering one uses, for example, as a characteristic dead load that load which only has a chance of being exceeded 5 per cent of the time. This load is now increased by 40 per cent in order to make the chance of it being exceeded extremely small. Similarly one may choose a characteristic moisture content such that it has a 15 per cent chance of being exceeded. In the case of pavements this does <u>not</u> mean that 15 per cent of the final pavement length will fail in the design period because of the following reasons:

(a) Failure in pavements is related to load application and even if sections of a pavement get saturated for short periods during its design life, only a small amount of the total load (E 80's) is likely to come onto the pavement during that time.

(b) If a pavement gets permanently saturated it will most probably fail, but this is likely to happen in isolated locations only. The design philosophy should be that such localized failures and their causes be repaired when they occur, rather than that the whole pavement be designed to withstand saturation.

(c) The associated subgrade strength loss with saturation will also be a factor determining whether failure will take place or not.

(d) It must also be remembered that the equations in Table 7.4 were developed from data representing failed as well as not failed, well-drained as well as poorly drained, cracked as well as uncracked, and rutted as well as not-rutted pavements, which means that for unfailed, well-drained pavements they will in any case be on the conservative side.

It is therefore recommended that the characteristic maximum moisture content be used as a design moisture content for pavement design, without a further increase or application of a further factor of safety. The following procedure should be followed to calculate the maximum or minimum characteristic moisture contents from the equations given in Table 7.4:

$$MC_{max/min} = PEMC \pm \left(\frac{S_{yx}}{\bar{x}}\right) (Z) \ (PEMC) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9.1)$$

where $MC_{max/min}$ = maximum or minimum characteristic moisture content
PEMC = predicted moisture content from equations in Table 7.4
S_{yx} = standard error or estimate
$\bar{x}$ = mean value [illegible] criteria
Z = statistical factor that will vary as the level of confidence varies and is available from statistical Z tables.

The ratio $S_{yx}/\bar{x}$ is given in Table 7.4 as the accuracy at the 85 per cent level because Z = 1,04 at the 85 per cent level.

Equation 9.1 may also be used to determine the chance that a specific material has of reaching satu[illegible]ng its design life, if the saturated moisture content is known,

$$Z = \frac{(MC_{sat} - PEMC) . \bar{x}}{S_{yx} . PEMC} \quad \ldots \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (9.2)$$

where MC_{sat} = saturated moisture content and the other variables have the same meaning as before.

Typical probabilities and the associated Z values are given in Table 9.1.

TABLE 9.1: PROBABILITIES AND ASSOCIATED Z VALUES (Pearson, 1924)

Z	0,84	1,04	1,28	1,64
Probability (%)	80	85	90	95

Example

Calculate the 90 per cent characteristic maximum and minimum moisture content as well as the chance that saturation will be reached for a material with the following properties: LL = 40; (% - 0,425) = 95 per cent; saturated moisture content = 26 per cent.

Using Table 7.4

$$PEMC = 0,42(LL)^{0,7}(\% - 0,425)^{0,3} - 3,9$$
$$= 17,9\ \% \quad (28\ \%\ \text{error})$$

Using equation 9.1 and Table 9.1

$$MC_{max} = 17,9 + (1,28)(0,28)(17,9)$$
$$= 24,3\ \%$$

$$MC_{min} = 17,9 - (1,28)(0,28)(17,9)$$
$$= 11,5\ \%$$

Using equation 9.2

$$Z = \frac{(26 - 17,9)}{(0,28)(17,9)}$$

Using Table 9.1, the probability of this happening is only about five per cent.

It should be clear that a well-defined method of predicting a design moisture content has been established. This method has already been incorporated in the different heave prediction methods. The NITRR (Weston, 1980) method of predicting percentage swell is:

$$\text{Swell}\ \% = 0,000411\{(LL)(\% - 0,425)\}^{4,17}.(P)^{-0,386}.(W_i)^{-2,33} \quad .. (9.3)$$

and the NBRI (Brackley, 1976) method is:

$$\text{Swel}\ \% = \{5,3 - \frac{147e}{PI} - \log_{10}P\}\{0,525(PI) + 4,08 - 0,85(W_i)\} \quad .. (9.4)$$

where LL = liquid limit

(% - 0,425) = percentage material finer than 0,425 mm

P = vertical overburden plus foundation pressure under which swell takes place (kPa)

W_i = initial moisture content

e = void ratio $(e = \frac{G\gamma_W}{\gamma_D} - 1)$

G = specific gravity (2,67)

γ_W = specific weight of water (9,81 kN/m^3)

γ_D = dry specific weight of the material (kN/m^3)

PI = plastic index

If applied as stated, both these methods will predict the heave from the initial moisture content to the saturated moisture content. Using the technique described above aximum characteristic moisture content may be predicted and, usir equation 9.3 or 9.4, the swell to saturation may first be calculated and then the swell from the maximum characteristic moisture content to saturation. The difference between these swells will give a more accurate estimate of the actual swell likely to happen under these conditions.

Unfortunately it is not so easy to incorporate this technique in the pavemen esign procedure without changing the CBR test method, which is not adv le without extensive laboratory investigations, because it would make future results incomparable with previous ones. At this stage only a modified interpretation of the soaked CBR test results will be proposed.

9.4.2 Estimating a design CBR

The easiest and most convenient way of incorporating moisture into the test procedure would simply be to compact the material in the mould at the design moisture content and penetrate it without soaking. This is unacceptable for reasons discussed before, although it has found practical application in Australia. Obviously the ideal method would be to firstly determine the compactive characteristics in the standard way, then to prepare different CBR moulds at the optimum moisture content and to subject each mould to a different moisture change. This will show the influence of moisture on the CBR for a specific compactive effort. The same procedure may be repeated for different compactive efforts to determine the influence of compactive effort on the CBR. Referring to a diagram showing the influence of both moisture and compactive effort on CBR, and knowing the specified compaction and design moisture content, will enable the pavement designer to make a good estimation of the design CBR. This approach is obviously also unacceptable, because it involves a lot of testing and is consequently very expensive.

The basic assumption made in developing an acceptable compromise is that the standardized test procedure should be retained for the time being and only the interpretation of the results modified. This was done, not because the standardized local method was considered a good one, but because it was feld that a change of test procedure from the start would not be accepted by the road authorities.

Haupt (1980) discussed the relative merits of different methods of CBR testing, but seeing that it was not one of the main aims of this study to determine the advantages and disadvantages of various methods of test, only the main ones will be summarized for the local test procedure.

The main disadvantage is clearly shown in Figure 9.2. Only the Mod. AASHTO compaction curve is determined and then duplicate samples are compacted at three different compaction efforts, but all at Mod. AASHTO optimum moisture content. These three pairs of samples are then soaked for 4 days and then penetrated. In this case only the Mod. AASHTO curve on Figure 9.2 (I), and Figure 9.2 (III) is reported and therefore it seems as if the CBR decreases as the density increases. The reason why this is possible is clear through the inclusion of Figure 9.2 (II). Normally the maximum CBR occurs at a moisture content slightly below optimum moisture content. For the Mod. AASHTO compaction, the sample is compacted at its optimum moisture content and the maximum CBR value is thus <u>not</u> measured. For the Standard Proctor compaction, the sample is compacted again at Mod. AASHTO optimum moisture content, which is lower than its optimum moisture content and which may coincide with the <u>maximum</u> CBR for Proctor compaction. Similarly the sample compacted at Standard AASHTO compaction and Mod. AASHTO optimum moisture content may lie between the above-mentioned points or on either side of them. Admittedly the above phenomenon is not a commonplace one, but it sometimes happens and the point is that one is

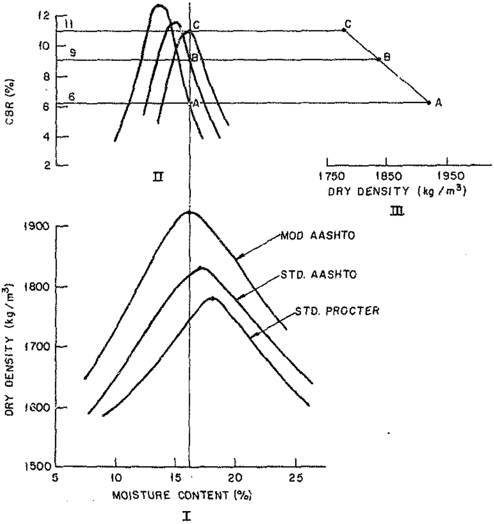


FIGURE 9.2

POSSIBLE CBR AND COMPACTION CURVES USING PRESENT METHODS not really comparing comparable things.

It should be clear that the method does not always show a true reflection of the influence of density. Another obvious disadvantage is that it does not represent either field moisture or density conditions in the majority of southern African locations. Finally, it does not show the compaction characteristics of a subgrade at the effort to be used in the field (all compaction curves at Mod. AASHTO effort).

The main advantage is that it is a standardized procedure. It also approximates the stress path occurring in the field and it does give an indication of the influence of density in most cases.

In order to arrive at a more realistic pavement design, the following procedure is suggested:

(a) Determine the characteristic maximum moisture content of the subgrade as set out in this dissertation.

(b) Determine the CBR of the sample in the standard way, but compare the moisture content after soaking with the characteristic moisture content. Remember that the sample moisture content after soaking but before penetration is an average moisture content and that the small layer on top that is actually penetrated may be wetter than this average.

(c) If the characteristic moisture content is higher than the soaked moisture content, then the soaked CBR must be used for design and quality control purposes.

(d) If the characteristic moisture content is less than the soaked moisture content, then the soaked CBR must be adjusted as follows:

$$CBR_u = 1{,}15(CBR_s) + 1{,}2 \quad \ldots\ldots \quad (9.5)$$

where CBR_u = unsoaked CBR at Proctor compaction

CBR_s = soaked CBR at Proctor compaction

This model has a correlation coefficient of 0,89 and a standard error of estimate of 5,3.

This method may also be used as an aid to better quality control. If a sample passes all other specifications except a CBR specification, equation 9.5 may be used, if the characteristic moisture content is less than the soaked value, to increase the measured CBR.

Fewer CBR determinations may be allowed if the equations reported in

Chapter 7 are used to predict the CBR. It should be remembered that those are Proctor CBR values, but they may be converted to Mod. AASHTO CBR values.

$$CBR = 0,37(CBR_M) \quad \ldots\ldots \quad (9.6)$$

where CBR = Proctor CBR

CBR_M = Mod. AASHTO CBR

This model has an error of 41 per cent at the 85 per cent level and was developed from CBR_M values ranging from 11 to 345 with a mean of 88 (673 cases were considered). More work is at present being carried out in this regard.

Appendix E shows how the principles explained here may be applied to practical problems.

It may also be very useful to know how Proctor optimum moisture content and maximum dry density may be converted to corresponding Mod. AASHTO values. These relationships are given in Chapter 7.

9.5 Conclusions

The following conclusions may be drawn from this chapter:

(a) The CBR test was originally devised purely as a classification test.

(b) After a road investigation in California it was adopted as a design basis and it was meant from the start to represent moisture conditions in the subgrade "under normal drainage and climatic conditions".

(c) Many countries have modified either the test method or the design procedure, because these moisture conditions do not exist in their subgrades.

(d) Locally the original test and design procedure are still applied. This study has shown that in the first place the moisture conditions in the subgrade are far from being generally saturated and, secondly, it has resulted in a reliable technique of predicting the design moisture content.

(e) There exists locally an obvious need to incorporate moisture rationally in the pavement design procedure and this study has provided guidelines for doing this. A characteristic moisture content is defined for design purposes. The characteristic moisture content (minimum or maximum) may be defined as that moisture content below or above which only a certain percentage of actual moisture content values will fall respectively. A method is outlined in detail whereby this moisture content may be calculated.

(f) More work should be done to develop methods for more accurately predicting a design CBR from material properties and specified density and moisture conditions. Preliminary guidelines in this regard are given in this chapter.

CHAPTER 10

REVIEW AND CONCLUDING REMARKS

CHAPTER 10

REVIEW AND CONCLUDING REMARKS

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CHAPTER 10

REVIEW AND CONCLUDING REMARKS

10.1 Review

Seeing that specific conclusions have been drawn at the end of each chapter, only the most important conclusions resulting from this study will be repeated here.

(a) The study was undertaken because there is an urgent need in many geotechnical and pavement engineering problems for a method of deriving the probable maximum or minimum moisture content likely to occur in the field.

(b) In order to understand the movement of water in soils, the energy potentials causing the movement should be understood, rather than that the water be categorized.

(c) Many conflicting observations about moisture trends have been made by researchers in the past. This is probably due to the many factors influencing a moisture regime, and it is very difficult to draw conclusions regarding a specific variable if all other influences are not carefully controlled. General agreement on some trends has nonetheless emerged.

(d) The largest moisture variations in pavements occur in the first two years after construction, after which relatively stable moisture conditions are achieved, specifically in the subgrade, except near the shoulders.

(e) Where the water table is shallow, it is a major factor influencing the moisture regime.

(f) Where the water table is deep, climate is a major factor influencing the moisture regime.

(g) Material type and density are important factors influencing the moisture regime for all levels of water table and climate. The moisture content increases with increasing plasticity and decreases with increasing density.

(h) Many instruments are available for measuring suction, but it was found in this study that the most reliable instruments for routine field measurements are tensiometers or psychrometers, depending on the suction range anticipated.

(i) In order to make the findings of this study reliable, a large number of samples, covering a wide range of material and climatic conditions, have been used in the analysis.

(j) When evaluating existing moisture prediction techniques, it became clear that the traditionally most prominent moisture predictor, plastic limit, is only accurate over limited ranges of plastic limit. It is thus not a practical predictor.

(k) None of the available models may be applied to local conditions without modifications, because of lack of accuracy.

(l) The optimum moisture content is the best predictor of the subgrade moisture conditions.

(m) The Atterberg limits and linear shrinkage adjusted for percentage material finer than 0,425 mm, correlate significantly better with material properties like CBR, potential heave, and in-situ moisture content than the unadjusted Atterberg limits and linear shrinkage. A provisional adjustment is proposed.

(n) A very important observation made during this study was that substantial temperature-induced pavement response variations occur on a daily basis in certain pavements. This has been confirmed during several experiments. The maximum suction and temperature usually occur around 14h00 and co-incide with the maximum curvature and thus maximum pavement strength. A minimum suction is reached at around 06h00, when the upper ± 400 mm of the pavement reaches a minimum temperature. Heavy loads applied during this time would cause most damage.

(o) This diurnal performance variation was not detected in regions where the rainfall exceeds 1 200 mm.

(p) The prediction models developed in this dissertation may be used with confidence in regions where the mean annual rainfall lies between

400 and 1 500 mm. They are not applicable where the rainfall is lower than 200 mm. They may also be used with either Proctor or Mod. AASHTO compactive effort, since accurate relationships between characteristics using these compactive efforts respectively, have been developed. It is felt that more accurate empirical moisture prediction techniques cannot be developed for local conditions, because of the size of the sample used and the nature of the data.

(q) Because reliable moisture prediction techniques are now available, it is proposed that the interpretation of the CBR test results be modified in order that they represent field conditions more closely. For this purpose a characteristic (minimum or maximum) moisture content (similar to the characteristic strengths and loads used in structural design) is defined for design purposes as that moisture content below or above which only a certain percentage of the actual moisture content values fall respectively. A method is given whereby this moisture content may be calculated as well as a preliminary method as to how this may be used to increase the CBR under certain circumstances.

(r) Although the moisture prediction techniques developed in Chapters 7 and 9 are considered suitable for most engineering applications, a preliminary rational method of predicting a design moisture content is also given. At this stage it is felt that it should be used mainly as a check to the empirical methods until it is more refined.

10.2 Suggestions for future research

It is clear that although this study has resulted in an acceptable moisture prediction technique, there is need for more work regarding the following aspects:

(a) A laboratory study should be carried out in order to determine qualitatively and quantitatively the influence of the osmotic water potential on the shear strength properties of a soil. It is known that it contributes to an increase in strength in membrane-like and in unsaturated situations, but the exact extent is not known.

(b) More work should be done to determine whether the proposed adjustement for Atterberg limits for percentage material finer than 0,425 mm does in fact relate best with other material properties such as heave potential and CBR. Consideration should be given to reporting the Atterberg limits in general in the adjusted form, rather than the unadjusted form. More work is also required in order to extended empirical relationships developed in this dissertation to other pavement layers (i.e. better quality materials).

(c) The major new field of research opened up by this study lies in the detailed investigation of daily temperature-induced performance variations. At this stage it has been confirmed that these trends do occur, but the influence of different pavement construction is not known. The influence of specific layers and climate has not been quantified. A laboratory study should be carried out in order to tie in this observation with the theory, so that it may be predicted at the design stage. More detailed in-situ field studies should be carried out, using mulitple-depth deflectometers, in order to quantify the influence of each pavement layer individually.

(d) More work should be done in order to quantify the influence of moisture and density on the CBR, so that the interpretation of the CBR test results may be modified with confidence. Prediction techniques should also be developed whereby the design CBR may be predicted from Atterberg limits, or some similar indicators.

APPENDIX A

ABBREVIATIONS, FORMAT OF DATA BASE AND FIELD MOISTURE SHEET

APPENDIX A

ABBREVIATIONS, FORMAT OF DATA BASE AND FIELD MOISTURE SHEET

This appendix contains the following information:

1 A comprehensive list of abbreviations used in equations, figures and tables in this dissertation.

2 The format of the final computer data base.

3 An example of a field sheet used in the moisture investigations.

1 Abbreviations

Variable	*Abbreviation*
Elevation	Elev
Surface drainage	Surdran
Road condition	Rcon
Crack condition	Crcon
Rut condition	Rutcon
Thickness of bitumen surface	Bitsur
Thickness of total cover	Totcover
Equilibrium moisture content	EMC
Grading modulus	GM
Transformed grading modulus $(GM)^{0,5}$	GMT
Liquid limit	LL
Adjusted liquid limit $(LL)^{0,7}$ $(\% - 0,425)^{0,3}$	LLT
Plastic limit	PL
Plasticity Index	PI
Adjusted plasticity index $(PI)(\% - 0,425)^{0,5}$	PIT
Bar linear shrinkage	LS
Adjusted bar linear shrinkage $LS\ (\% - 0,425)^{0,7}$	LST

Variable	Abbreviation
Sand equivalent	SE
Proctor maximum dry density	MDD
MAASHTO maximum dry density	MDD_M
Standard AASHTO maximum dry density	MDD_S
Proctor optimum moisture content	OMC
MAASHTO optimum moisture content	OMC_M
Standard AASHTO optimum moisture content	OMC_S
Maximum soaked California Bearing Ratio when the five moulds used for the determination of the Proctor compaction characteristics are penetrated	CBR
Transformed California Bearing Ratio $(CBR)^{0,6}$	CBRT
Maximum soaked California Bearing Ratio when the five moulds used for the determination of the MAASHTO compaction characteristics are penetrated	CBR_M
Percentage material finer than 0,425 mm	(% - 0,425)
Percentage material finer than 0,075 mm	(% - 0,075)
Mean annual rainfall	MAR
Weinert's climatic N-value	N
Transformed climatic N-value $(N)^{-1}$	NT
Thornthwaite's climatic I-index	I

2 Format of final computer data base

For each site a record of information is stored in three lines, the contents of each line being defined as follows:

2.1 Contents of the first line

2.1.1 Location of the hole in the road: for example N003-1/34.0

This is a string character which must always contain 11 characters. The first character is alphabetic and can only be J, N, P or K.

2.1.2 Elevation

Elevation is described by the following figures:

1 = flat
2 = fill
3 = side fill
4 = cut
5 = vlei

or a combination of any two above, so that it may be represented by a maximum figure of 54 which means vlei + cut.

2.1.3 Surface drainage

Surface drainage may be described by the figures 1, 2 or 3 according to the following code:

1 - good (water runs off pavement and into veld)
2 - fair (water runs off pavement but stands next to road)
3 - poor (water stands on pavement)

2.1.4 Road condition

Road condition is indicated by the figures 1, 2 or 3 with the following meaning:

1 - not failed
2 - border line
3 - failed

This description of the road condition is related to the immediate vicinity of the hole but not necessarily to the exact spot where drilling took place. This is unfortunate because the pavement may be failed at one spot and two meters away it may be unfailed. It is also difficult to infer whether the subjective definition of failure used here can be extended to the subgrade.

2.1.5 Crack condition

Here there are only two possibilities:

1 - not cracked
2 - cracked

If the condition is uncracked (1), the next four items are 0,001 to indicate that no information is available for these items. If the condition is cracked (2), on the other hand, further information is supplied according to the following code:

1 = crocodile
2 = block
3 = longitudinal
4 = transverse

If only crocodile cracking is present the first item will be 1 and the rest 0,001. If the crocodile cracking and longitudinal cracking are present the first two items will read 1 and 3, followed by 0,001 and 0,001. The crack condition is thus always described by five items of which four could be 0,001 if no cracks are present.

2.1.6 Rut condition

Here there are only two possibilities:

1 = not rutted
2 = rutted

2.1.7 Thickness of bitumen surfacing

This is indicated by a figure equal to the thickness in mm.

2.1.8 Total cover

This is indicated by a figure equal to the total thickness in mm.

2.1.9 Moisture content of the subgrade

This is indicated by a percentage and seeing that all the roads sampled were older than five years, it is assumed that this percentage is the equilibrium moisture content.

2.2 Contents of the second line

The second line contains 10 items of information obtained from laboratory and road tests. They are:

2.2.1 Grading modulus

This is indicated by a figure between 0 and 3.

2.2.2 Liquid limit

This is indicated by a figure less than 100.

2.1.7 Thickness of bitumen surfacing

This is indicated by a figure equal to the thickness in mm.

2.1.8 Total cover

This is indicated by a figure equal to the total thickness in mm.

2.1.9 Moisture content of the subgrade

This is indicated by a percentage and seeing that all the roads sampled were older than five years, it is assumed that this percentage is the equilibrium moisture content.

2.2 Contents of the second line

The second line contains 10 items of information obtained from laboratory and road tests. They are:

2.2.1 Grading modulus

This is indicated by a figure between 0 and 3.

2.2.2 Liquid limit

This is indicated by a figure less than 100.

2.2.3 Plasticity index

This is indicated by a figure less than 80.

2.2.4 Linear shrinkage

This is indicated by a figure less than 40.

2.2.5 Sand equivalent

This is indicated by a figure less than 100.

2.2.6 Classification

This is a single string character of variable length, used to describe the soil classification. For example:

A-2-4(0) where the figure in brackets is the group index.

2.2.7 Maximum density

This item consists of a single figure giving the value of the maximum subgrade dry density in kg/m^3. An upper limit of 3 000 cannot be exceeded.

2.2.8 Optimum moisture content

Optimum moisture content is expressed as a percentage and it indicates the gravimetric moisture content required to achieve maximum density at Proctor compaction for the subgrade.

2.2.9 Maximum CBR

This is the value of the maximum CBR obtained on the CBR versus moisture content curve for the subgrade. The limiting value is 150.

2.2.10 Deflection

This is the deflection value as obtained from the deflectograph expressed in mm.

2.3 Contents of the third line

2.3.1 Percentage material finer than 0,425 mm.

This item is expressed in a percentage and is the percentage of subgrade material that passes the 0,425 mm sieve.

2.3.2 Percentage material finer than 0,075 mm

This item is expressed as a percentage and is the percentage of subgrade material that passes the 0,075 mm sieve.

2.3.3 Mean annual rainfall

This item is expressed in decimetres and has a maximum value of 20.

2.3.4 Weinert's N-value

This item is expressed as an absolute value between 0 and 10.

2.3.5 Thornthwaite's I-value

This item is expressed as an absolute value between 40 and 100.

Thus in total ±26 500 items of information were stored away and analysed. The statistical analysis was done on all the data at first, but later points that could be left out with a fair amount of certainty were left out. Only the results of the final analysis are reported here, although the others are available from NITRR.

NITRR MOISTURE INVESTIGATION ON EXISTING ROADS.

BOREHOLE INFORMATION. SHEET 1

ROAD P6/1 POSITION KM 42,6 ALTITUDE > 2000 m

REGION BAPSFONTEIN DATE 2/4/1978

ELEVATION	FLAT X	FILL	SIDE FILL	CUT	VLEI

SURFACE DRAINAGE	GOOD X	FAIR	POOR

SLOPE EROSION	NONE	<150mm	>150mm
FILL	X		
CUT	X		

ROAD CONDITION AT VICINITY OF BOREHOLE	NOT FAILED X	FAILED

SURFACE CONDITION AT VICINITY OF BOREHOLE

NOT CRACKED				
CRACKED X	CROCODILE	BLOCK X	LONGIT.	TRANSVERSE X
RUT DEPTH 20 mm				

SAMPLE NO. BAG	TIN	CORE	SAMPLE DEPTH	LAYER THICK-NESS	MATERIAL DESCRIPTION	CONDITION FRESH OR BRITTLE FOR SURFACING, DENSE OR LOOSE, WET OR DRY FOR LAYERS
				50	Bitumen seal	Brittle
RR51	A1		180	200	Sandstone gravel in matrix of red sandy silt	Moist and dense
RR52	A2		320	150	Red intact transported sandy clay	Moist and firm
RR53	A3		600	-	Brownish intact silty clay	Wet and soft

GENERAL REMARKS

e.g. Gum trees adjacent to road etc.

MOISTURE CONTENT Sample No.	A1	A2	A3
Wet WT.	268	241	301
Dry WT.	241	193	222
Diff	27	48	79
% Moisture	11,2	24,9	35,6

APPENDIX B

DETAILS OF REGRESSION ANALYSES TO VALIDATE EXISTING MODELS FOR LOCAL CONDITIONS

APPENDIX B

DETAILS OF REGRESSION ANALYSES TO VALIDATE EXSITING MODELS FOR LOCAL CONDITIONS

This Appendix contains the details of the regression analyses carried out on the variables used in existing models in an attempt to modify them to suit local conditions better. The data are presented in tabular form and the relevant models can be identified as follows:

(a) Swanberg and Hansen model with no limits on PL (equation 5.1).

(b) Swanberg and Hansen model with PL between the limits 15 and 30 (equation 5.1).

(c) Madrid Transport and Soil Mechanics Laboratory model using LL (equation 5.5).

(d) Madrid Transport and Soil Mechanics Laboratory model using (%-0,075) (equation 5.6).

(e) Brodie's model (equation 5.12).

(f) Gawith's model incorporating mean annual rainfall (equation 5.14).

(g) Gawith's model without mean annual rainfall (equation 5.15).

(h) Arulunandan's model (equation 5.18).

(i) Van der Merwe's model (equation 5.2p).

Table B1 shows the mean, standard deviation and number of cases of the variables involved in the specific model.

Table B2 gives the correlation coefficients of the predictors with the dependent variable equilibrium moisture content as well as inter-predictor correlations for the number of cases specified.

Table B3 gives the results of the regression analyses on the various existing models. The models tabulated in this table are always of the general form:

$$\text{Equilibrium moisture content} = B_1 \cdot \text{variable}_1 \pm B_2 \cdot \text{variable}_2 \pm B_n \cdot \text{variable}_n$$

Table B4 gives statistics about the regression formulae tabulated in Table B3.

Table B5 constitutes a summary table of the regression analysis.

TABLE B1: MEAN, MEAN S STANDARD DEVIATION AND NUMBER OF CASES

Variable		Mean	Standard deviation	Cases
(a)	EMC	10,9020	5,5973	967
	PL	16,0352	4,3881	967
(b)	EMC	12,9141	6,0226	505
	PL	18,6475	3,4299	505
(c)	EMC	10,8964	5,5901	970
	PL	28,8474	11,3414	970
(d)	EMC	10,5028	5,5454	1052
	%-0,075	35,6749	17,4037	1052
(e)	EMC	10,5103	5,5213	1059
	OMC	12,8024	4,2486	1059
(f)	EMC	10,9253	5,6176	951
	%-0,075	37,1157	17,5212	951
	$\%-0,075^2$	1684,2408	1543,5075	951
	PL	15,9989	4,3527	951
	LL	28,8601	11,3026	951
	MAR	6,8767	1,2701	951
(g)	As in (f) but without MAR			951
(h)	EMC	10,9253	5,6176	951
	%-0,075	37,1157	17,5212	951
	PL	15,9989	4,3527	951
(i)	EMC	10,9039	5,5946	962
	PL	16,0260	4,3767	962
	MDD	1881,4345	170,0058	962

TABLE B2: CORRELATION COEFFICIENT MATRIX

N	Predictor	EMC	PL	%-0,075	%-0,075	LL	MDD
(a) 967	PL	,60064					
(b) 505	PL	,49894					
(c) 970	LL	,77147					
(d) 1052	%-0,075	,67809					
(e) 1059	OMC	,83564					
(f) 951	%-0,075	,66616					
	$\%-0,075^2$	,66736					
	PL	,60513		,34891	,41125		
	LL	,77454	,82471	,57536	,62637		
	MAR	,17576	,24483	,20832	,18376	,09608	
(g) 951	%-0,075	,66616					
	$\%-0,075^2$	,66736					
	PL	,60513		,34891	,41125		
	LL	,77454	,82471	,57536	,62637		
(h) 951	%-0,075	,66616					
	PL	,60513		,34891			
(i) 962	PL	,60007					-.59834
	MDD	-,81180					

TABLE B3: VARIABLES IN THE EQUATION AND RELEVANT STATISTICS

Variable		B	Std. error B	Significance	Beta	95 per cent confidence interval
(a)	PL	,76615	,03283	0	,60064	,83058 : ,70173
	Constant	-1,38342	,54576	,011	-	-2,45444 : -,31240
(b)	PL	,87608	,06785	0	,49894	,74277 : 1,00938
	Constant	-3,42253	1,28642	,008	-	-5,94995 : -,89512
(c)	LL	,38025	,01009	0	,77147	,36047 : ,40003
	Constant	-,07289	,31242	,816	-	-,68599 : ,54020
(d)	%-0,075	,21606	,00723	0	,67809	,20188 : ,23024
	Constant	2,79483	,28685	0	-	2,23197 : 3,35769
(e)	OMC	1,08596	,02196	0	,83564	1,04266 : 1,12904
	Constant	-3,39249	,29615	0	-	-3,97360 : -2,81138
(f)	LL	,39649	,02036	0	,61665	,26652 : ,34645
	%-0,075	,21633	,02624	0	,67473	,16483 : ,26784
	$\%-0,075^2$	-,00141	,00031	0	-,38675	-,00201 : -,00081
	MAR	,19795	,08966	,027	,04476	,02200 : ,37391
	PL	,01193	,04700	,800	,00925	-,08031 : ,10418
	Constant	-5,12959	,79355	0	-	-6,68691 : -3,57228
(g)	LL	,29282	,01944	0	,58915	,25467 : ,33097
	%-0,075	,22757	,02580	0	,70978	,17694 : ,27820
	$\%-0,075^2$	-,00148	,00030	0	-,40772	-,00208 : ,00089
	PL	,05069	,04369	,246	,03928	-,03505 : ,13643
	Constant	-4,28370	,69640	0	-	-5,05028 : -2,91712
(h)	%-0,075	,16611	,00701	0	,51808	,15235 : ,17987
	PL	,54769	,02822	0	,42437	,49230 : ,60307
	Constant	-4,00238	,45034	0	-	-4,88616 : -3,11860
(i)	PL	,22765	,02917	0	,17809	,17041 : ,28490
	MDD	-,02371	,00075	0	-,70524	-,24682 : -,02173
	Constant	50,92057	1,73657	0	-4,00455	47,51266 : 54,32849

TABLE B4: STATISTICS ABOUT THE REGRESSION FORMULAE TABULATED IN TABLE B3

Multiple correlation coefficient R		R^2	Adjusted R^2	Standard deviation
(a)	,60064	,36077	,36011	4,47746
(b)	,49894	,24894	,24745	5,22456
(c)	,77147	,59517	,59475	3,55859
(d)	,67809	,45981	,45929	4,07765
(e)	,83564	,69829	,69800	3,03422
(f)	,82611	,68245	,68077	3,17397
(g)	,82511	,68081	,67946	3,18046
(h)	,77584	,60193	,60109	3,54807
(i)	,82425	,67939	,67872	3,17111

TABLE B5: SUMMARY TABLE OF REGRESSION ANALYSIS

Step		Variable entered removed	Significance of variable	Multiple correlation coefficient R	R^2	R^2 change	Simple correlation coefficient R
(a)	1	PL	0	,60064	,36077	,36077	,60064
(b)	1	PL	0	,49894	,24894	,24894	,49894
(c)	1	LL	0	,77147	,59517	,59517	,77147
(d)	1	%-0,075	0	,67809	,45981	,45981	,67809
(e)	1	OMC	0	,83564	,69829	,69829	,83564
(f)	1	LL	0	,77454	,59991	,59991	,77454
	2	%-0,075	0	,82012	,67260	,07269	,66616
	3	%-0,075^2	0	,82484	,68036	,00776	,66736
	4	MAR	,013	,82609	,68243	,00207	,17576
	5	PL	,800	,82611	,68245	,06062	,60513
(g)	1	LL	0	,77454	,59991	,59991	,77454
	2	%-0,075	0	,82012	,67260	,07269	,66616
	3	%-0,075^2	0	,82484	,68036	,00776	,66736
	4	PL	,246	,82511	,68081	,00045	,60513
(h)	1	%-0,075	0	,66616	,44376	,44376	,66616
	2	PL	0	,77584	,60193	,15816	,60513
(i)	1	MDD	0	-,81180	,6590	,6590	-.81180
	2	PL	0	,82425	,6794	,0204	,60007

Example of how the tables should be used

Say for example one wants to find out how applicable the Van der Merwe model is to local conditions. Equation (5.20)

$$PEMC = \left(\frac{PL}{0,001(\rho_d) + 0,17}\right)$$

was proposed by him. Results of the testing of this model with local data are reported under Section 5.2.14. The results of the regression done on local data to modify this model are contained in these Tables B1 to B5 under the sub-heading (i).

Table B1 shows that the results of 967 boreholes were taken into account in developing this model. The mean and standard deviation of the relevant variables for the 967 cases are also given.

Table B2 simply shows the regression correlation coefficients that are calculated before the actual regression started.

Table B3 shows the best linear regression equation that can be developed from this data, viz.:

$$EMC = 0,227(PL) - 0,023(OMC) + 50,92$$

The standard error in each of the coefficients B is given together with the significance level of the regression coefficient of each variable (probability that regression coefficient is zero), taking the variables included earlier into account. The 95 per cent confidence intervals for each coefficient and the standardized coefficient beta, which is very useful for comparison purposes, are also included.

Table B4 shows that the multiple correlation coefficient for the above equation is 0,82425, that the percentage variation that can be explained by this model, R^2, is 68 per cent. R^2 is adjusted for the number of variables in the model. The standard deviation of the model is also given.

Table B5 shows how the multiple correlation coefficient, R, as an indication of accuracy of prediction, improves with the entrance or removal of every variable. The significance level of the regression coefficient of each variable is also given.

APPENDIX C

VARIATION OF SUBGRADE MOISTURE
CONTENT WITH DIFFERENT PARAMETERS

APPENDIX C

VARIATION OF SUBGRADE MOISTURE CONTENT WITH DIFFERENT PARAMETERS

This Appendix contains plots to show how the subgrade moisture content varies with different parameters. The dotted line shows the best line through the data, and the table number in brackets refers to the table in the text (chapter 5) that shows the moisture content broken down by the same parameter plotted in the figure.

FIGURE C1 (Table 5.8)

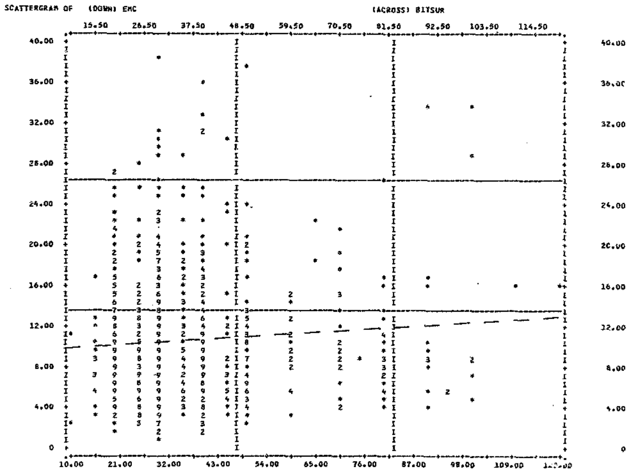


FIGURE C2 (Table 5.9)

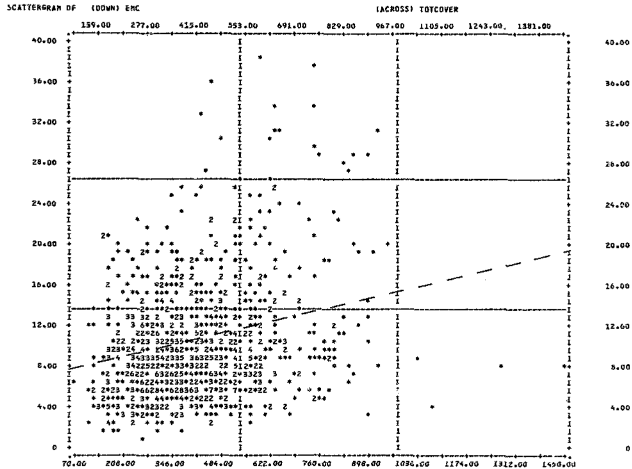


FIGURE C3 (Table 5.10)

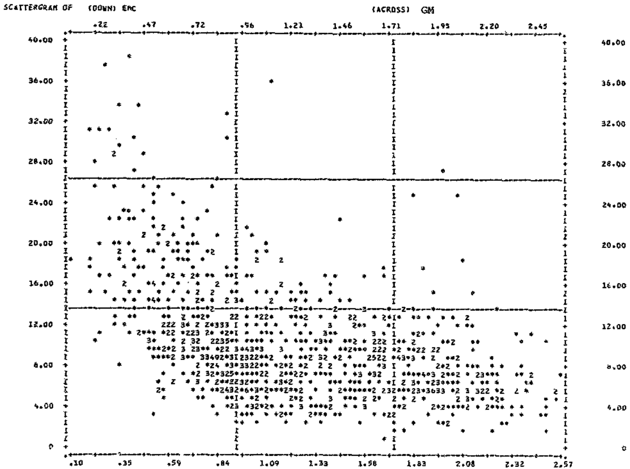


FIGURE C4 (Table 5.11)

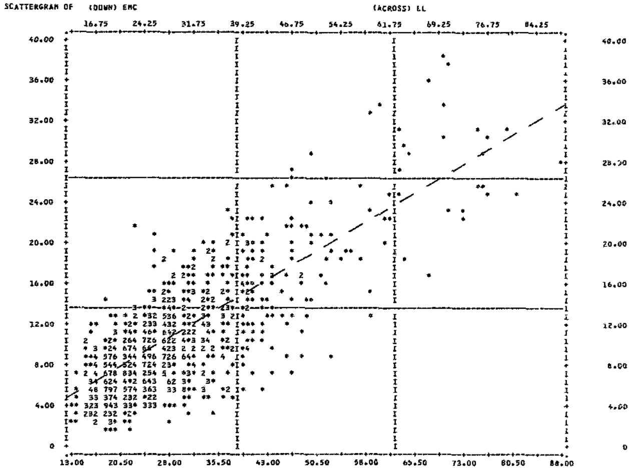


FIGURE C5 (Table 5.12)

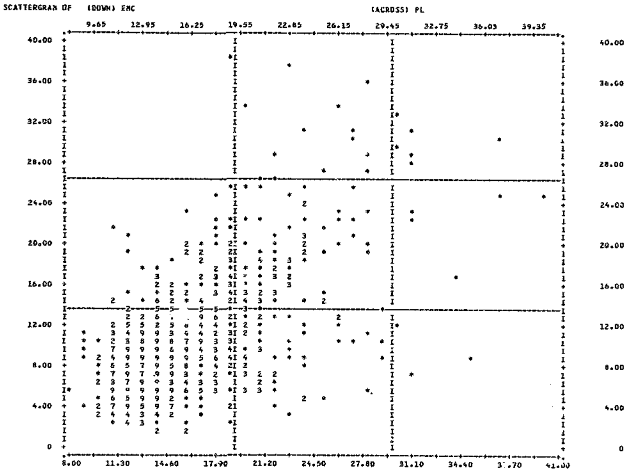


FIGURE C6 (Table 5.13)

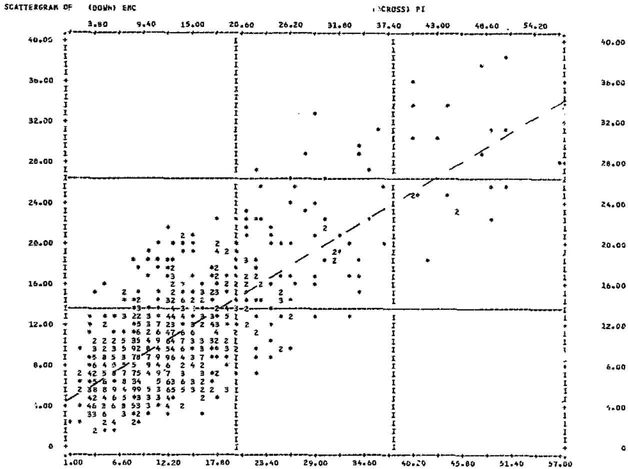


FIGURE C1 (Tabla 5.14)

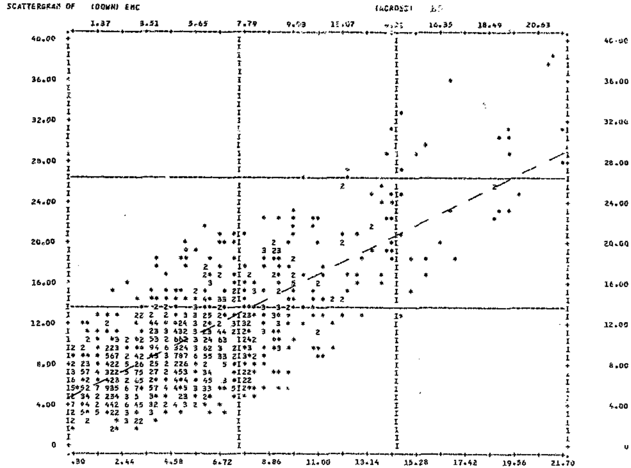


FIGURE C8 (Table 5.15)

SCATTERGRAM OF (DOWN) EHC (ACROSS) SE

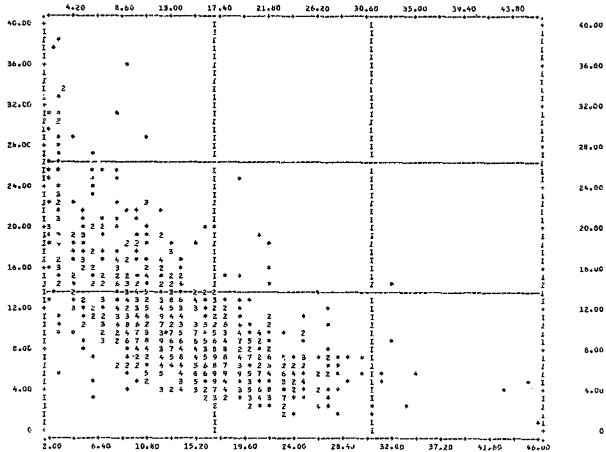


FIGURE C9 (Table 5.16)

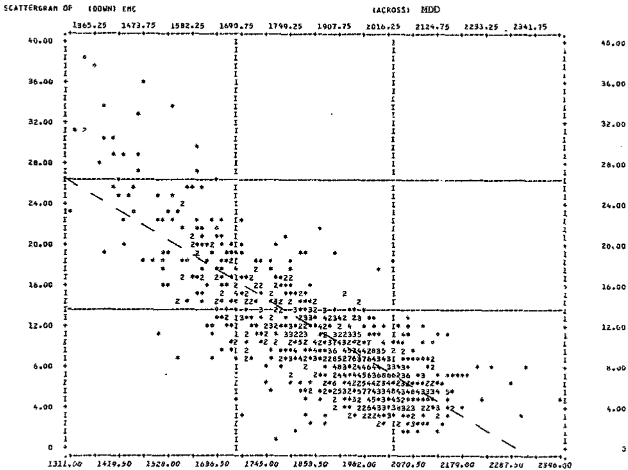


FIGURE C10 (Table 5.17)

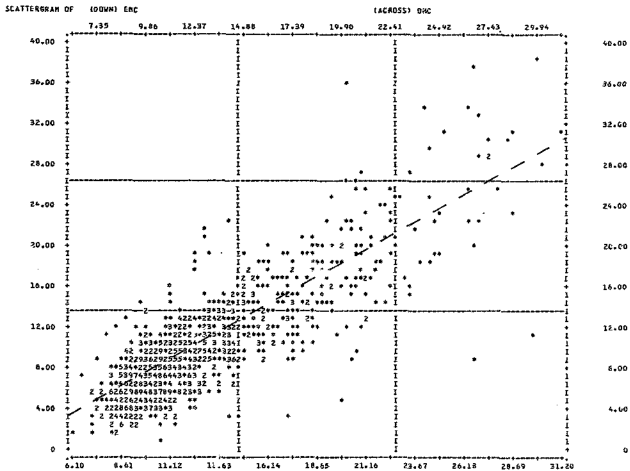


FIGURE C11 (Table 5.18)

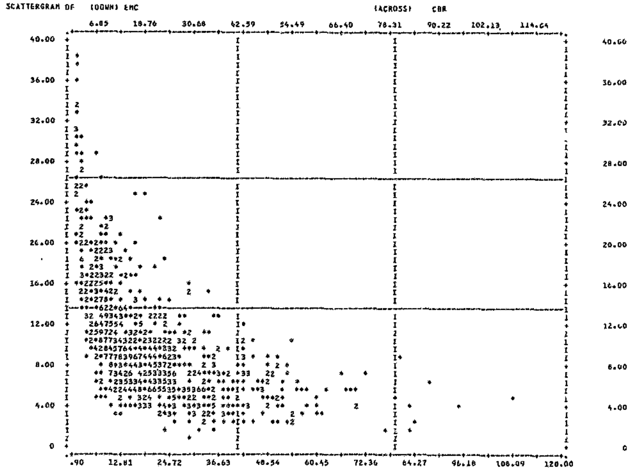


FIGURE C12 (Table 5.19)

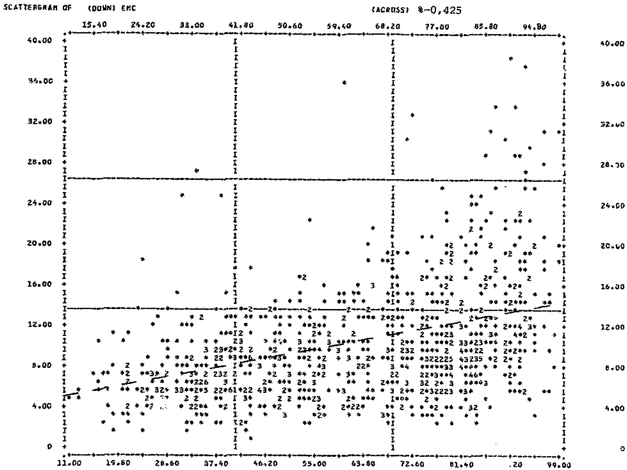


FIGURE C13 (Table 5.20)

SCATTERGRAM OF (DOWN) EMC (ACROSS) %-0,075

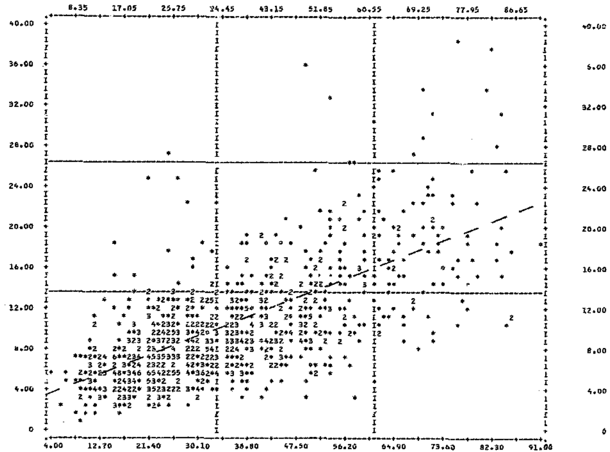


FIGURE C14 (Table 5.21)

SCATTERGRAM OF (DOWN) EMC (ACROSS) MAR

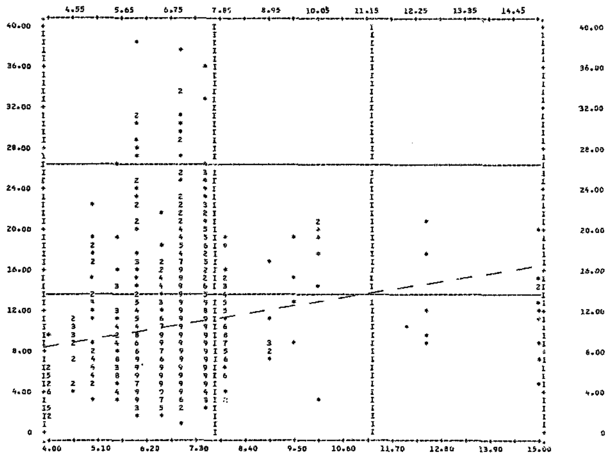


FIGURE C15 (Table 5.22)

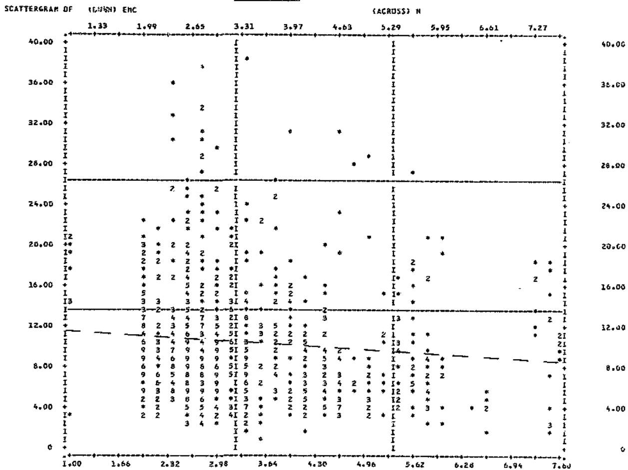
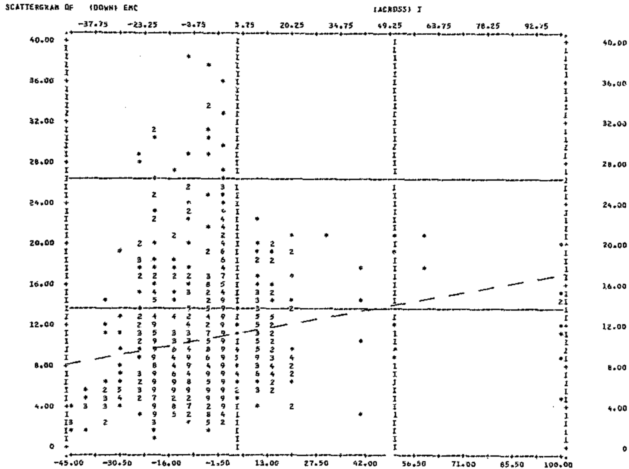


FIGURE C16 (Table 5.23)



APPENDIX D

DETAILS OF REGRESSION ANALYSES ON LOCAL DATA

APPENDIX D

DETAILS OF REGRESSION ANALYSES ON LOCAL DATA

This appendix contains the details of the regression analyses carried out in order to develop the best moisture prediction models for local conditions. The main features of this process is described in chapter 7 and the abbreviations used in the tables are explained in appendix A.

The different regressions reported on in this appendix may be identified as follows:

Model A General linear regression on all cases and parameters (Section 7.3.1)

Model B General linear regression on all cases and ratio and interval parameters (Section 7.3.1)

Model C Linear regression on all parameters limiting optimum moisture content values to less than 13 (Section 7.3.3(a))

Model D Linear regression on all parameters limiting optimum moisture content values to greater than or equal to 13 (Section 7.3.3(b))

Model E Linear regression on all ratio and interval parameters limiting optimum moisture content values to greater than or equal to 13 (Section 7.3.3(b))

Table D1 comprises five tables, one for each model. It gives descriptive statistics about each regression.

Similarly, Table D2 is divided into five tables and it gives the inter-parameter correlation coefficients of all variables involved.

Table D3 gives the results of the regression analyses for the various models.

Table D4 gives descriptive statistics about the analyses tabulated in Table D3.

Table D5 summarizes the stepwise regression analysis for each model.

TABLE D1 : MEAN, STANDARD DEVIATION AND NUMBER OF CASES OF THE VARIABLES INVOLVED IN THE SPECIFIC REGRESSION

MODEL A

Variable	Mean	Standard Dev	Cases
EMC	11,0103	5,6676	914
ELEV	1,6991	2,1874	914
SURDRAN	2,0974	,7116	914
RCON	1,4158	,7326	914
CRCON	1,4114	,4924	914
RUTCON	1,2659	,4420	914
BITSUR	34,3818	15,3537	914
TOTCOVER	409,7484	169,2458	914
GM	1,2367	,5701	914
LL	29,0810	11,3780	914
PI	12,9628	8,1760	914
LS	5,9447	3,6132	914
SE	13,7597	6,0647	914
MDD	1879,1740	171,0334	914
OMC	13,2166	4,3532	914
CBR	18,0208	13,0125	914
% - 0,425	62,5591	21,8198	914
% - 0,075	37,2429	17,5495	914
MAR	6,6550	1,2689	914
N	3,3340	1,2125	914
I	-5,2954	16,5234	914
PIT	104,1022	77,9655	914
LLT	35,7413	11,3443	914
PL	16,1182	4,3322	914
NT	,3358	,1155	914
GMT	,9931	,2932	914
CBRT	,2238	,1090	914
LST	110,4911	85,4542	914

TABLE D1 (continued)

MODEL B

Variable	Mean	Standard Dev	Cases
EMC	10,9895	5,6678	918
GM	1,2373	,5701	918
LL	29,0566	11,3613	918
PI	12,9499	8,1627	918
LS	5,9391	3,6074	918
SE	13,7771	6,0675	918
MDD	1879,5479	170,9731	918
OMC	13,2038	4,3507	918
CBR	18,0476	13,0104	918
% - 0,425	62,5338	21,8226	918
% - 0,075	37,2026	17,5616	918
MAR	6,8475	1,2748	918
N	3,3415	1,2192	918
I	-5,3813	16,5697	918
PIT	103,9800	77,8507	918
LLT	35,7173	11,3316	918
PL	16,1068	4,3263	918
NT	,3353	,1156	918
GMT	,9929	,2932	918
CBRT	,2236	,1089	918
LST	110,3658	85,3402	918

TABLE D1 (continued)

MODEL C

Variable	Mean	Standard Dev	Cases
EMC	7,9724	2,8869	547
ELEV	1,7495	2,4750	547
SURDRAN	2,0896	,7040	547
RCON	1,3766	,6959	547
CRCON	1,4406	,4969	547
RUTCON	1,2377	,4260	547
BITSUR	34,0128	14,8907	547
TOTCOVER	393,2907	160,8988	547
GM	1,4346	,5238	547
LL	23,1974	5,3423	547
PI	9,0329	3,9105	547
LS	4,1308	1,9302	547
SE	16,4177	5,0631	547
MDD	1985,7166	85,9346	547
OMC	10,4400	1,3784	547
CBR	23,0435	13,7011	547
% - 0,425	56,1225	21,3607	547
% - 0,075	28,4918	11,1330	547
MAR	6,6325	,9992	547
N	3,3996	1,2156	547
I	-7,6782	12,8425	547
PIT	65,1837	29,0168	547
LLT	29,3167	4,5867	547
PL	14,1645	2,7427	547
NT	,3263	,0963	547
GMT	,8819	,1742	547
CBRT	,1742	,0544	547
LST	65,9282	32,6263	547

TABLE D1 (continued)

MODEL D

Variable	Mean	Standard Dev	Cases
EMC	15,5381	5,7746	367
ELEV	1,6240	1,6695	367
SURDRAN	2,1090	,7236	367
RCON	1,4741	,7815	367
CRCON	1,3678	,4829	367
RUTCON	1,3079	,4623	367
BITSUR	34,9319	16,0237	367
TOTCOVER	434,2779	178,3963	367
GM	,9418	,5056	367
LL	37,8501	12,3094	367
PI	18,8[illegible]	9,2981	367
LS	8,6482	3,8422	367
SE	9,7981	5,2165	367
MDD	1720,3760	140,3708	367
OMC	17,3550	3,9677	367
CBR	10,5346	6,9606	367
% - 0,425	72,1526	18,7823	367
% - 0,075	50,2861	17,2724	367
MAR	7,1866	1,5306	367
N	3,2362	1,2030	367
I	-1,7439	20,3443	367
PIT	162,1089	90.9373	367
LLT	45,3168	11,6627	367
PL	19,0300	4,6249	367
NT	,3501	,1382	367
GMT	1,1589	,3508	367
CBRT	,2976	,1268	367
LST	176,9104	96,1075	367

TABLE D1 (continued)

MODEL E

Variable	Mean	Standard Dev	Cases
EMC	15,5255	5,7718	368
GM	,9406	,5055	368
LL	37,8234	12,3033	368
PI	18,8071	9,2888	368
LS	8,6429	3,8383	368
SE	9,7959	5,2095	368
MDD	1720,4293	140,1831	368
OMC	17,3459	3,9661	368
CBR	10,5310	6,9514	368
% - 0,425	72,2065	18,7852	368
% - 0,075	50,3098	17,2548	368
MAR	7,1875	1,5286	368
N	3,2345	1,2018	368
I	-1,7391	20,3168	368
PIT	162,0333	90,8249	368
LLT	45,3024	11,6501	368
PL	19,0163	4,6260	368
NT	,3502	,1380	368
GMT	1,1596	,3506	368
CBRT	,2976	,1267	368
LST	176,8610	95,9811	368

TABLE D2 : MATRICES OF CORRELATION COEFFICIENTS BETWEEN THE TABLED VARIABLES

MODEL A

	EMC	ELEV	SURDRAN	RCON	CRCON	RUTCON	BITSUR	TOTCOVER	GM	LL	PI	LS	SE
ELEV	-,03903												
SURDRAN	,04230	-,13456											
RCON	,18119	,01253	,18908										
CRCON	-,05773	,01844	,07311	,28142									
RUTCON	,13395	-,04405	,03600	,51739	,13606								
BITSUR	,08563	-,06213	,04812	-,02289	,03730	,00165							
TOTCOVER	,29515	-,02432	-,20815	-,03007	-,03871	,04767	,06136						
GM	-,50921	,11635	-,06552	-,01456	-,00355	-,02689	-,03819	-,16067					
LL	,77471	,01066	-,03615	,11685	,00285	,06344	,02452	,22202	-,31082				
PI	,75696	-,04123	,01964	,10170	,01333	,02971	,04423	,21578	-,38554	,95423			
LS	,75686	-,03576	,00573	,09580	,00802	,03281	,06062	,19940	-,37577	,94819	,97850		
SE	-,62509	,08261	-,05840	-,08027	-,01715	-,06481	,00824	-,21752	,68641	-,52450	-,57999	-,58823	
MDD	-,81899	,03669	-,06740	-,10502	,00923	-,07351	-,05015	-,20468	,68765	-,77934	-,76144	-,76126	,69541
OMC	,83671	-,01049	-,00127	,09798	-,02277	,08860	,04199	,22667	-,52912	,87806	,82274	,83236	-,63173
CBR	-,55482	,06039	,00400	-,0538[illegible]	,01883	-,04721	-,01376	-,17693	,69445	-,47478	-,52507	-,[illegible]4422	,70413
% - 0,425	,45234	-,11893	,09115	,00360	-,01073	,02636	,02413	,15749	-,98130	,21597	,29754	,28272	-,66898
% - 0,075	,67139	-,11228	,03126	,04206	,01263	,04730	,06828	,18222	-,87644	,57846	,61391	,63154	-,77132
MAR	,17775	,02314	-,01589	-,01168	-,04469	,05219	,05133	,08977	-,16319	,10084	,00618	,01418	-,13681
N	-,10819	-,02942	-,00943	,00354	,15889	,02418	-,11245	-,00664	,02867	-,07300	-,05199	-,06517	-,05109
I	,17049	,01390	,03691	-,0120[illegible]	-,08064	,05425	-,01324	,02694	-,12512	,03482	-,06044	-,05780	-,10011
PIT	,79437	-,06803	,04272	,08995	,01288	,03695	,05145	,23906	-,57345	,90555	,96706	,94256	-,66820
LLT	,83310	-,03861	,00777	,10262	-,00022	,06746	,03504	,24108	-,62699	,92900	,92377	,91881	-,70615
FL	,60612	,10582	-,13199	,11[illegible]35	-,01768	,11055	-,01907	,17588	-,08873	,82550	,61892	,64362	-,20296
MT	,13515	,06638	,05408	,05760	-,07661	,01234	,05145	-,02820	-,04009	,06241	,00380	,01999	,03680
GMT	,57444	-,08646	,05246	,01506	,00191	,02239	,05505	,18306	-,88652	,47389	,53327	,52289	-,65469
CBRT	,76050	-,07555	,04554	,10129	-,00134	,07407	,04533	,28675	-,64990	,72458	,78684	,77840	-,72231
LST	,80296	-,07289	,04215	,08209	,00779	,04007	,06730	,23005	-,62343	,88011	,93232	,94200	-,69757

MODEL A (continued)

MDD	OMC	CBR	%-0,425	%-0,075	MAR	N	I	PIT	LIT	PL	NT	GMT	CBRT
-,91895													
,64916	-,55186												
-,62203	,44677	-,66044											
-,81633	,75046	-,70244	,81152										
-,22123	,24022	-,18101	,13805	,21549									
,08058	-,09430	,07348	-,00081	-,06036	-,59473								
-,18415	,17483	-,13098	,11636	,14195	,85534	-,47341							
-,83188	,85529	-,58908	,49573	,74799	,03415	-,04061	-,03018						
-,90193	,92403	-,65024	,54687	,80651	,14006	-,06172	,07562	,96169					
-,60980	,75339	-,25602	,00567	,36068	,25317	-,09361	,20552	,55324	,69651				
-,11623	,11862	-,06833	,00658	,04955	,68909	-,85531	,61372	-,00310	,05819	,15674			
-,72271	,64526	-,57961	,84931	,89831	,12287	-,02544	,07056	,69713	,71602	,23819	,00241		
-,78971	,74729	-,72256	,59829	,75251	,07323	-,02314	,01959	,85766	,84505	,41806	-,00456	,67944	
-,85192	,87047	-,61719	,54495	,80281	,04953	-,04810	-,01794	,98255	,96384	,55199	,00828	,73957	,86611

TABLE D2 (continued)

MODEL B

		TH	LL	PI	LS	SE	MDD	OMC	CBR	%-0,425	%-0,075	MAR	N
GM	-												
LL	[illegible]	.095											
PI	,75703	,38553	,95425										
LS	,75691	-,37604	,94819	,97652									
SE	-,62655	,68641	-,52522	-,58047	-,58873								
MDD	-,81919	,68817	-,77923	-,76140	-,76130	,69658							
OMC	,83715	-,52950	,87802	,82268	,83230	-,63290	-,91899						
CBR	-,55605	,69385	-,47551	-,52572	-,54485	,70513	,64990	-,95262					
% - 0,425	,45280	-,98135	,21655	,29796	,28342	-,66939	-,62300	,44753	-,66035				
% - 0,075	,67200	-,87649	,57851	,61387	,63160	-,77225	-,81701	,75083	-,73302	,81211			
MAR	,18374	-,16669	,10478	,00985	,01784	-,14368	-,22598	,24473	-,18567	,14227	,22127		
N	-,11498	,03385	-,07669	-,05534	-,06848	-,04205	,08691	-,10040	,07893	-,00672	-,06807	-,60040	
I	,17574	-,12831	,33220	-,05698	-,05433	-,10622	-,18853	,17931	-,13515	,12001	,14740	,85685	-,47970
PIT	,79437	-,57365	,90553	,96701	,94259	-,66866	-,83199	,85529	-,58969	,49630	,74804	,03812	-,04465
LLT	,83322	-,62729	,92891	,92370	,91880	-,70677	-,90202	,92404	-,65082	,54758	,80666	,14417	-,06661
PL	,60675	-,08916	,82565	,61917	,64379	-,28406	-,60976	,75354	-,25683	,00649	,36098	,25526	-,09700
NT	,13965	-,04376	,06504	,00632	,02252	,03086	-,12040	,12266	-,07222	,01074	,05496	,69161	-,85599
GMT	,57430	-,88688	,47369	,53310	,52296	-,65526	-,72332	,64532	-,57998	,84990	,89845	,12697	-,03083
CBRT	,76099	-,64996	,72491	,78703	,77866	-,72301	-,79014	,74779	-,72311	,59472	,75369	,07815	-,02860
LST	,80284	-,62385	,87999	,93220	,94195	-,69801	-,85208	,87041	-,61775	,54574	,80291	,05357	-,05230

MODEL B (continued)

I	PTT	VT	PL	NT	GMT	CBKT
-,02648						
,07954	,96171					
,20784	[illegible]	,69662				
,61672	-,00009	,06166	,15887			
,07433	,69714	,71613	,23812	,00632		
,02407	,85786	,84538	,41874	-,00065	,67957	
-,01417	,98255	,96386	,55209	,01142	,73976	[illegible]

TABLE D2 (continued)

MODEL C

	EMC	ELEV	SURDRAN	RCON	CRCON	RUTCON	BITSUR	TOTCOVL	GM	LL	PI	LS	SE
ELEV	-,07013												
SURDRAN	,044[illegible]	[illegible]											
RCON	,16763	[illegible]	[illegible]270										
CRCON	-,10822	,02436	[illegible]8068	,24490									
RUTCON	,10659	-,02682	-,03447	,44503	,10143								
BITSUR	,06368	-,04648	,05388	-,04006	,09478	,00674							
TOTCOVER	,21717	-,00841	-,22566	-,01845	-,03718	,07821	,05466						
GM	-,29845	,14948	-,10389	,01163	-,01190	-,01240	-,00901	-,10219					
LL	,24008	,08409	-,03847	,09342	,03754	,00992	-,04221	-,02963	,34238				
PI	,28936	-,01712	,07211	,06745	,06887	-,07066	-,01627	-,07526	,12943	,36902			
LS	,28207	,02407	,02833	,05681	,07297	-,06637	-,00462	-,08238	,18744	,88340	,96011		
SE	-,43854	,11367	-,06010	-,03771	-,03398	-,05120	,04538	-,11210	,57072	-,02879	-,22192	-,20112	
MDD	-,44213	,05805	-,12576	-,06825	-,03872	-,01897	-,03060	-,04065	,66519	-,08292	-,22267	-,17867	,48316
OMC	,51479	,03639	-,08769	,08182	-,02072	,07265	-,02256	,11379	-,20960	,52267	,48965	,50039	-,33411
CBR	-,44518	,05600	,00556	-,01261	-,00331	-,00883	,00858	-,09624	,61353	-,22890	-,36937	-,35600	,60351
% - 0,425	,28968	-,14907	,09877	-,01457	-,00647	,02477	-,00615	,10384	-,98387	-,39126	-,18798	-,24483	-,58499
% - 0,075	,43256	-,17426	,09415	,00867	,04585	,02860	,02569	,09175	-,87412	-,07899	,13067	,09757	-,71149
MAR	,28048	,01974	-,16271	-,06401	-,16209	,10872	,14052	,20302	-,17400	,02014	-,06018	-,06204	-,14355
N	-,17101	-,02109	,05675	,03524	,21282	,03412	-,11264	-,06739	-,05481	-,10234	-,07309	-,08315	-,10389
I	,36823	,00911	-,06862	-,03756	-,22229	,13328	,02757	,13707	-,19128	-,01991	-,10419	-,11440	-,17479
PIT	,43859	-,09311	,13491	,06617	,07241	-,06076	-,01108	-,03748	-,35005	,61310	,85973	,79343	-,51245
LLT	,49954	-,04198	,00830	,08355	,03592	,02934	-,03755	,04202	-,47835	,63044	,70538	,67337	-,55682
PL	,05508	,18820	-,27514	,06631	-,02507	,12008	-,05902	,04959	,48235	,70877	,26691	,35179	,26034
NT	,19649	,06208	-,05367	-,01516	-,17592	,00983	,09207	,07981	,03847	,10832	,04913	,05401	,11936
GMT	,30487	-,15668	,13469	-,01086	,03169	,01844	,01259	,08245	-,96664	-,29901	-,09259	-,15098	-,56967
CBRT	,49260	-,09459	,09426	,07577	,04871	,01432	,04979	,09321	-,63163	,20177	,40867	,36903	-,62280
LST	,46181	-,07835	,11285	,05462	,08300	-,05407	,00085	-,03541	-,41237	,54943	,77170	,77641	-,55313

MODEL C (continued)

MDD	OMC	CBR	%-0,425	%-0,075	MAR	N	I	PIT	LLT	PL	NT	GMT	CBRT
-,57739													
,60821	-,49260												
-,63759	,18165	-,58077											
-,68569	,37305	-,67790	,84167										
-,20736	,27193	-,23034	,18111	,19359									
,06646	-,11776	,07169	,05915	,05186	-,61909								
-,25712	,22034	-,20299	,20957	,19734	,74669	-,46711							
-,53504	,56467	-,61832	,29211	,36069	,01848	-,02512	-,00031						
-,63271	,68121	-,71652	,43097	,64949	,14677	-,03491	,13329	,88781					
,15597	,31993	,08078	-,49410	-,34017	,12502	-,09513	,10977	-,03159	,22227				
-,09425	,13206	-,07936	-,04678	-,07914	,63837	-,93448	,49841	-,00190	,04389	,14094			
-,65015	,21744	-,56714	,96060	,87935	,16140	,06564	,19344	,36779	,47199	-,45041	-,06105		
-,64150	,50525	-,84487	,59375	,73970	,14832	-,03651	,14472	,70845	,71858	-,18968	,04154	,62332	
-,56486	,58479	-,64862	,35555	,63262	,03344	-,02300	,01483	,95692	,88781	-,03010	-,01083	,42711	,73276

TABLE D2 (continued)

MODEL D

	EMC	ELEV	SURDRAN	RCON	CRCON	RUTCON	BITSUR	TOTCOVER	GM	LL	PI	LS	SE
ELEV	,01147												
SURDRAN	,04870	-,10168											
RCON	,20644	,05744	,18378										
CRCON	,05737	-,00080	,06479	,34751									
RUTCON	,12087	-,07615	,12811	,60075	,20115								
BITSUR	,11166	-,09646	,03952	-,00614	-,03913	-,01007							
TOTCOVER	,35885	-,04825	-,19274	-,06182	-,02117	-,01005	,06290						
GM	-,40977	,04228	-,06464	,01860	-,07935	,03423	-,05643	-,15189					
LL	,73960	,00523	-,04540	,11534	,08928	,02542	,03860	,33886	-,34152				
PI	,71125	-,05506	-,01860	,09261	,08050	,00720	,06621	,34650	-,42199	,94617			
LS	,71250	-,08359	-,02637	,08473	,06920	[illegible]	,09902	,33735	-,45179	,92981	,97312		
SE	-,46408	,01262	-,06147	-,07458	-,11481	,00432	,00551	-,27712	,64616	-,48147	-,54234	-,56263	
MDD	-,75508	-,01780	-,05929	-,10218	-,10588	-,02533	-,05501	-,29609	,62159	-,80635	-,75549	-,76021	,58958
OMC	,74392	,01307	,01275	,08312	,10506	,03826	,06100	,29506	-,50093	,85879	,77239	,78236	-,51486
CBR	-,47294	,04408	,03139	-,06772	-,06294	-,02272	-,02303	-,28014	[illegible]	-,47112	-,56177	-,59353	,72899
% - 0,425	,36986	-,03807	,08301	-,03212	,05676	-,04350	,04733	,15642	-,97784	,25693	,35322	,37697	-,64066
% - 0,075	,47013	-,06751	-,03048	-,00198	,10135	-,02578	,09943	,18460	-,89586	,48681	,51832	,56786	-,64794
MAR	-,06041	,05053	,11603	,00576	,10093	-,02738	-,03791	-,05211	,00679	-,07435	-,19278	-,19750	,08243
N	-,03132	-,05591	-,10436	-,02879	,06494	,02312	-,10844	,09452	,08745	-,00568	,01719	,00470	-,10076
I	-,06369	,03735	,13173	-,01316	,07521	-,03717	-,05987	-,10486	,07142	-,13503	-,25067	-,26181	,14609
PIT	,72507	-,07680	,01021	,07249	,08896	,00503	,07365	,35419	-,58680	,90212	,97326	,95354	-,61302
LIT	,75884	-,02430	-,00828	,08704	,10179	,10555	,05472	,34358	-,62220	,94306	,93033	,93012	-,63021
PL	,53857	,12461	-,08344	,12079	,07580	,15318	-,03039	,20528	-,06058	,75933	,50785	,51568	-,19111
NT	,03905	,09339	,16240	,11373	,04092	-,00053	,00790	-,15434	-,05498	-,05160	[illegible]	-,11049	,09987
GMT	,44156	-,02843	-,00222	-,02254	,05129	-,03995	,07410	,19813	-,88003	,44518	,50755	,52411	-,57580
CBRT	,67762	-,07860	,02170	,08755	,05297	,05468	,03004	,39526	-,58333	,68472	,75868	,76417	-,67271
LST	,72115	-,10088	,01775	,05960	,07875	,00493	,10289	,34103	-,66736	,85874	,92434	,95255	-,64753

MODEL D (continued)

MDD	OMC	CBR	%-0,425	%-0,075	MAR	N	I	FIT	LLT	PL	NT	GMT	CBRT
-,90427													
,55663	-,44154												
-,36600	,42084	-,71806											
-,68602	,64258	-,68450	,82282										
-,02315	,07443	,13218	-,05436	,05714									
,03146	-,05642	-,01377	-,03421	-,10174	-,60092								
,02453	,01147	,19475	-,09998	-,04846	,91423	-,49949							
-,80281	,79882	-,62645	,52764	,64628	-,17674	,01141	-,23891						
-,88303	,88626	-,64258	,55134	,70391	-,07251	-,01844	-,14049	,95693					
-,62727	,73286	-,12450	-,02628	,25361	,18968	-,04970	,14459	,44437	,63964				
-,03993	,04356	,08180	-,01695	,02762	,72029	-,80155	,68192	-,11504	-,04169	,11051			
-,64512	,59401	-,58033	,84313	,88488	-,03725	-,03821	-,10920	,65393	,66428	,16447	-,04587		
-,70788	,62777	-,78302	,55986	,59699	-,14314	,05290	-,19463	,80942	,78034	,29715	-,12721	,55819	
-,81728	,80769	-,66723	,60797	,72831	-,17085	-,00164	-,23920	,97520	,95194	,42685	-,10015	,71620	,81756

TABLE .. continued)

MODEL E

	EMC	GM	LL	PI	LS	SE	MDD	OMC	CBR	$-0,425	$-0,075	MAR	N
GM	- 40602												
LL	,74006	-,33890											
PI	,71150	-,42012	,94613										
LS	,71273	-,44991	,92878	,97313									
SE	-,46333	,64583	-,48070	-,54191	-,56221								
MDD	-,75470	,62056	-,80593	-,75539	-,76012	,58949							
OMC	,74439	-,49785	,85905	,77255	,78249	-,51399	-,90369						
CBR	-,47208	,73354	-,47027	-,56127	-,59303	,72901	,55652	-,44065					
$ - 0,425	,36667	-,97786	,25402	,35107	,37481	-,64010	-,56472	,41736	-,71749				
$ - 0,075	,46845	-,89580	,48512	,51724	,56677	-,64790	-,68557	,64057	-,68449	,82273			
MAR	-,06080	,00628	-,07473	-,19299	-,19770	,08235	-,02307	,07386	,13206	-,05368	,05740		
N	-,03012	,08861	-,00453	,01793	,00543	-,10050	,03125	-,05513	-,01349	-,03567	-,10239	-,60095	
I	-,06382	,07114	-,13509	-,25070	-,26183	,14604	,02456	,01126	,19469	-,09958	-,04832	,91421	-,49842
PIT	,72501	-,58534	,90189	,97321	,95391	-,61280	-,80280	,79864	-,62618	,52589	,64555	-,17688	,01185
LIT	,75896	-,62024	,94296	,93037	,93016	-,62983	-,88293	,88619	-,64213	,54904	,70284	-,07274	-,01778
PL	,53961	-,05777	,75981	,50838	,51617	-,19034	-,62666	,73347	-,12373	-,02932	,25162	,18876	-,04[illegible]3
NT	,03846	-,03555	-,05209	-,12357	-,11079	,09976	-,03983	,04294	,08166	-,01620	,02796	,72033	-,80153
GMT	,43914	-,88023	,44276	,50587	,52244	-,57564	-,64429	,59119	-,58023	,84339	,88492	-,03679	-,03927
CBRT	,67754	-,58200	,68464	,75870	,76419	-,67251	-,70789	,62770	-,78277	,55820	,59636	-,14327	,05325
LST	,72289	-,66614	,85836	,92442	,95244	-,64740	-,81729	,80730	-,66707	,60647	,72776	-,17093	-,00137

<u>MODEL E</u> (continued)

I	PIT	LLT	PL	NT	GMT	CSRT
-,23895						
-,14055	,95692					
,14410	,44451	,63977				
,68192	-,11522	-,04198	,10958			
-,10893	,65268	,66261	,16180	-,04530		
-,19467	,80946	,78037	,29743	-,12737	,55713	
-,23923	,97519	,95186	,42670	-,10027	,71519	,81758

Table D3 gives the results of the regression analyses for the various categories. The models tabulated in this table are always of the general form:

$$EMC = B_1 \times variable_1 \pm B_2 \times variable_2, \ldots\ldots\ldots \pm B_n \times variable_n$$

TABLE D3 : VARIABLES IN THE EQUATION AND RELEVANT STATISTICS

Variable	B	Std.error B	Significance	Beta
MODEL A				
OMC	,78535359	,32505397E-01	0	,6032189
CBRT	15,622144	1,2986643	0	,3004372
RCON	,70[illegible]10949	,12901161	0	,0916607
(CONSTANT)	[illegible]5063	,33976202	0	
MODEL B				
OMC	[illegible]046	,33250069E-01	0	,5893068
CBRT	16,[illegible]75011	1,3185016	0	,32[illegible]3541
NT	3,3122361	,83058430	0	,0675776
(CONSTANT)	-3,9857244	,38565662	0	
MODEL C				
OMC	,77702963	,[illegible]5103594E-01	0	,3710098
SE	-,15557353	,20255982E-01	0	-,2728487
I	,53677918E-01	,77166056E-02	0	,2387881
(CONSTANT)	2,8265489	,95419806	,003	
MODEL D				
LLT	,20795598	,33869645E-01	0	,4200014
MDD	-,15250489E-01	,28181281E-02	0	-,3707157
RCON	,97547279	,23848168	0	,1320073
(CONSTANT)	30,912864	6,2720774	0	
MODEL E				
LLT	,24863131	,35333875E-01	0	,5018527
MDD	-,18043320E-01	,28692647E-02	0	-,4382317
% - 0,075	-,61785909E-01	,15385571E-01	0	-,1847106
(CONSTANT)	38,412648	6,3705102	0	

Table D4 gives statistics about the analyses tabulated in Table D3

TABLE D4 : STATISTICS ABOUT ANALYSES TABULATED IN TABLE D3

Mode'	Multiple correlation coefficient R	R^2	Adjusted R^2	Standard Error of Estimate
A	0,86592	0,74981	0,74899	2,83953
B	0,86404	0,74656	0,74573	2,85798
C	0,63133	0,39525	0,39525	2,24501
D	0,79113	0,62589	0,62279	3,54657
E	0,79063	0,62509	0,62200	3,54857

Table D5 constitutes a summary table of the regression analyses

TABLE D5 : SUMMARY TABLE OF REGRESSION ANALYSES

Step	Variable Entered	Significance of variable	Multiple correlation coefficient R	R^2	R^2 change	Simple R
MODEL A						
1	OMC	0	,83671	,70009	,70009	,83671
2	CBRT	0	,86111	,74151	,04142	,76050
3	RCON	0	,86592	,74981	,00831	,18119
4	TOTCOVER	0	,86947	,75597	,00616	,29515
5	CRCON	0	,87233	,76097	,00499	-,05773
MODEL B						
1	OMC	0	,83715	,70082	,70082	,83715
2	CBRT	0	,86148	,74215	,04133	,76099
3	NT	0	,86404	,74656	,00441	,13965
4	MAR	,006	,86526	,74868	,00212	,18374
5	I	0	,87020	,75725	,00857	,17574
MODEL C						
1	OMC	0	,51479	,26500	,26500	,51479
2	SE	0	,58735	,344[illegible]8	,07998	-,43854
3	I	0	,63133	,39857	,05359	,36823
4	CBRT	0	,64680	,41835	,01978	,49260
5	RCON	0	,65998	,43557	,01722	,16763
MODEL D						
1	LLT	0	,75884	,57584	,57584	,75884
2	MDD	0	,78016	,60864	,03280	-,75508
3	RCON	0	,79113	,62589	,01724	,20644
4	CBRT	0	,80043	,64069	,01480	,67762
5	GM	0	,81121	,65807	,01738	-,40877
MODEL E						
1	LLT	0	,75896	,57602	,57602	,75896
2	MDD	0	,78005	,60848	,03246	-,75470
3	%-0,075	0	,79063	,62509	,01661	,46845
4	CBRT	0	,80262	,64419	,01910	,67754
5	OMC	,001	,80936	,65507	,01088	,74439

APPENDIX E

EXAMPLE OF HOW MOISTURE PREDICTION TECHNIQUES MAY BE USED IN PRACTICE

APPENDIX E

EXAMPLE OF HOW MOISTURE PREDICTION TECHNIQUES MAY BE USED IN PRACTICE

It is proposed that a road carrying a design load of $0{,}5.10^6$ E80's (lightly trafficked) be built in the Pretoria area. The following routine soil test results from the subgrade material were received from a laboratory:

% -2,0 mm	= 99 %;	% -0,425 mm	= 95 %
% -0,075 mm	= 80 %;	LL	= 35
PL	= 17;	LS	= 7

A soaked CBR test at the specified density (Proctor) was also done and the CBR found to be equal to 6. The mould moisture content after soaking was 29 per cent.

Determine for this road, at the 85 per cent level, the following:

(a) the design moisture content and the design CBR;

(b) the chances that the saturated moisture content will be reached in the field;

(c) the total potential heave;

(d) the Proctor and Mod. AASHTO soaked and unsoaked CBR's;

(e) check the design moisture content using the rational method.

Calculations

(a) The design moisture content is equal to the maximum characteristic moisture content.

$$EMC = 0{,}42(LL)^{0{,}7}(\% - 0{,}425)^{0{,}3} - 3{,}9 \qquad \text{From eqn (7.21)}$$
$$= 15{,}9 \text{ per cent}$$

$$MC_{max} = 15{,}9 + (1{,}04)(0{,}28)(15{,}9) \qquad \text{From eqn (9.1) and Table 7.4}$$
$$= 20{,}5 \text{ per cent}$$

The maximum probable moisture content at the 85 per cent level is thus 20,5 per cent. This is significantly lower than the soaked value of 29 per cent. The design CBR may thus with confidence be increased from the measured soaked value of 6, i.e.:

$$\begin{aligned} CBR_u &= 1{,}15\ (CBR_s) + 1{,}2 \qquad \text{From eqn (9.5)} \\ &= 1{,}15\ (6) + 1{,}2 \\ &= 8{,}1 \end{aligned}$$

or if the CBR test above had <u>not</u> been done, see (d).

(b) The chances that saturation (assumed to be the same as the mould moisture content after penetration) will be reached in the field may be determined by applying eqn (9.2) and statistical Z tables:

$$Z = \frac{(MC_{sat} - PEMC)\ \bar{x}}{(Syx)(PEMC)} \qquad \text{From eqn (9.2)}$$

$$= \frac{(29 - 15{,}9)}{(0{,}28)(15{,}9)}$$

$$= 2{,}94$$

Using Z tables, the probability of this happening is only 1,6 per cent.

(c) In order to predict the total heave the initial moisture content must be known. It is reasonable to assume that this will be close to the Proctor optimum moisture content for the subgrade. If this is not known it may be estimated from eqn (7.44).

$$\begin{aligned} OMC &= 0{,}3(LL)^{0{,}7}\ (\% - 0{,}425)^{0{,}3} + 0{,}2(PL) - 1 \qquad \text{From eqn (7.44)} \\ &= 0{,}3(35)^{0{,}7}\ (95)^{0{,}3} + 0{,}2(17) - 1 \\ &= 16{,}6 \text{ per cent} \end{aligned}$$

A maximum moisture content change of 20,5 - 16,6 = 3,9 per cent will thus occur and cause a heave. If the layer thickness is not known, calculate the potential swell for the first 1 m to decide whether more accurate investigations are warranted.

Percentage swell to saturation for first 1 m:

$$= 0{,}000411 \ \{(LL)(\% - 0{,}425)\}^{4{,}17} \ . \ (P)^{-0{,}386} \ . \ (W_1)^{-2{,}33}$$

From eqn (9.3)

P for the first meter $= \rho g h$

Assume ρ to be the Proctor maximum dry density + 16,6 per cent water. If the Proctor maximum dry density is not known it may be estimated from eqn (7.43).

$$MDD = 10{,}3(PI) - 20{,}5(LL)^{0{,}7} \ (\% - 0{,}425)^{0{,}3} + 2\ 477$$

From eqn (7.43)

$$= 10{,}3(18) - 20{,}5(35) \ (95)^{0{,}3} + 2\ 477$$

$$= 1\ 695 + 0{,}166 \ . \ 1\ 695 \ kg/m^3$$

$$= 1\ 975 \ kg/m^3$$

Thus $P = (1\ 975)(9{,}81)(0{,}5)$

$$= 9{,}6 \ kPa$$

Thus percentage swell to saturation

$$= 0{,}000411 \ \{(35)(0{,}95)\}^{4{,}17} \ (9{,}6)^{-0{,}386} \ . \ (16{,}6)^{-2{,}33}$$

$$= 0{,}55 \text{ per cent}$$

Percentage swell from characteristic maximum moisture content to saturation:

$$= 0{,}000411 \ \{(35)(0{,}95)\}^{4{,}17} \ (9{,}6)^{-0{,}386} \ . \ (20{,}5)^{-2{,}33}$$

$$= 0{,}33 \text{ per cent}$$

Thus physical swell caused by the first meter is (0,55 - 0,33 per cent)·10 = 2 mm.

The swell due to any lower layers will decrease rapidly because the confining pressure increases rapidly with depth. At depth greater than 7 m the contribution of swelling soil to a physical movement at the surface is very small. Heave is thus unlikely to be a problem in this case.

(d) Even if no CBR test had been done, the design CBR may still be estimated as follows. The Proctor soaked CBR may be estimated by:

$$CBR = 2{,}1\,(e^{GM}) - 23\log\{(LS)(\% - 0{,}425)^{0{,}7}\} + 54$$

From Table 7.6

$$= 2{,}1\,(e^{0{,}26}) - 23\log\{(7\ )(95)^{0{,}7}\} + 54$$

$$= 5{,}4$$

The corresponding unsoaked value is 7,5 (eqn (9.5)).

The Mod. AASHTO soaked CBR is related to the Proctor CBR by

$$CBR_M = 2{,}7\,(CBR)$$ From eqn (9.6)

$$= 14{,}6$$

The corresponding unsoaked value is 18 (eqn (9.5)).

(e) The design moisture content may also be determined using the rational procedure explained in chapter 8. From Figure 8.8 the characteristic matric section for Pretoria is approximately 3,0 (2,8 in Table 8.3). Using the value of 3,0 , which would be used in the proposed method, a moisture content of 18 per cent is read off Figure 8.10. This is lower than the 20,5 per cent predicted from the empirical methods but is close enough to serve as a check. The latter value should be used for design purposes.

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