# THE PREDICTION OF MOISTURE CONTENT IN UNTREATED PAVEMENT LAYERS AND AN APPLICATION TO DESIGN IN SOUTHERN AFRICA

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**SYNOPSIS**

Methods have been developed to predict pavement moisture content in South Africa so that it may be used in pavement design.

The data used were taken from existing surfaced roads in the Transvaal, Cape Province, and Natal. The effects of materials and climate on moisture content were found. Models were developed to predict equilibrium moisture content and the ratio of equilibrium to optimum moisture content for all pavement layers and climates of South Africa. The seasonal and spatial variation of moisture content was studied. The minimum width of sealed shoulder to substantially reduce the probability of trafficking in the edge zone of seasonal moisture variation was defined. A new suction/climate relationship was developed to predict equilibrium suction on a regional basis, and a new climate map of Thornthwaite's moisture index was produced to complement this.

Laboratory and field data were used to quantify the variation of strength with moisture for pavement materials. A new unsoaked CBR criterion was proposed for subgrade classification and design. The costs of pavements designed using the proposed unsoaked criteria were compared to the costs of those designed using the present soaked criterion. A Bayesian probability analysis approach was used to find the total discounted costs over the analysis period, and unsoaked design was shown to be less expensive than the soaked design for all but very high predicted moisture contents.

**CONTENTS** **Page**

**LIST OF ABBREVIATIONS**

| Abbreviation | Quantity |
|---|---|
| ANOVA | Analysis of variance |
| CBR | California Bearing Ratio |
| CBRomc | CBR at OMCm |
| CBRs | Soaked CBR |
| CBRu | Unsoaked CBR: CBR at some moisture content less than soaked (usually field moisture content) |
| cf. | Compared with |
| DCP | Dynamic cone penetrometer |
| df | Degrees of freedom |
| E | Modulus of elasticity |
| EMC | Equilibrium moisture content |
| EMC/OMCm | Ratio of equilibrium to Mod. AASHTO optimum moisture content |
| E80 | 80 kN single axle load |
| F | F-test result; Factor of safety |
| Im | Thornthwaite's moisture index |
| K | Layer variable: 1 for basecourse, 0 for subbase or subgrade |
| LL | Liquid limit |
| LS | Linear shrinkage |
| MAR | Normal annual rainfall (generally the long term 30 year average) |
| mc | Moisture content |
| mc/OMCm | Ratio of moisture content to Mod. AASHTO optimum moisture content |
| MDDm | Maximum dry density at Mod. AASHTO compaction |
| MDDp | Maximum dry density at Proctor compaction |
| ME80s | Million equivalent 80 kN single-axle loads |
| Mr | Resilient modulus from repeated load triaxial testing |
| n | Number of samples |
| N | Weinert's N value |
| na | Not available/applicable |
| OMCm | Optimum moisture content at Mod. AASHTO compaction |
| OMCp | Optimum moisture content at Proctor compaction |
| PI | Plasticity index |
| PL | Plastic limit |

| | |
|---|---|
| P425 | Percentage material passing a 0,425 mm sieve |
| P75 | Percentage material passing a 0,075mm sieve |
| R,r | Correlation coefficient |
| s.d. | Standard deviation |
| SEE | Standard error of estimate |
| SXX | Corrected sum of squares |
| t | Student's t |
| z | Standardised score or standard normal variable |

## 1 INTRODUCTION

One of the main factors determining the performance of a pavement is the strength of materials in that pavement. Strength in turn is significantly affected by moisture, and this bulletin discusses the moisture regime of pavements, prediction of moisture, and its use in design.

In South Africa, materials classification of unstabilised and modified soils and gravels is carried out using the soaked CBR test to measure strength. Most pavement design methods use subgrade resilient modulus or CBR determined in the soaked condition. However it is known that the moisture regime of many pavements in South Africa is drier than this. In other countries, considerable savings are made in the road industry by design using subgrade strength at expected field moisture condition (such as NAASRA, 1979). Such savings should be possible in South Africa once the pavement moisture regime is understood and can be predicted.

What was needed for this was to quantify the pavement moisture regime by measuring moisture in existing pavements in all layers, all climatic areas, and all seasons. From this, methods could be developed to predict a probable moisture content. The predicted moisture contents could then be used to find the expected field strength of the materials which would be used for design purposes. Since unbound materials are mostly stronger in an unsoaked state than a soaked one, lower quality materials could be used or thinner pavements designed, and the construction costs would be reduced. However the risk of premature distress when using unsoaked design must be balanced against the cost savings.

In the first major database used (Burrow, 1975), 1100 sites across the Transvaal Province of South Africa were selected by systematic experimental design with samples taken every kilometre along selected roads. At each site a hole was augered centred at 1,4 metres in from the lane edge and the subgrade sampled. Laboratory testing was done using Proctor compaction were applicable.

In the second major database used, 96 sites across the Cape and Natal Provinces were selected by a stratified random experimental design to include different climates and pavements with cracked and uncracked surfacings, good and poor surface drainage, sandy and clayey subgrades. At each site, four sample holes spaced 25 metres apart were augered at 1,4 metres in from the lane edge. Moisture and geotechnical samples were taken of each pavement layer from the moisture equilibrium zone. Undisturbed equilibrium suction samples were taken at 450mm below the surfacing. The equilibrium samples were taken at what was estimated to be the wettest time of year (Emery, 1981). At eight selected sites, seasonal moisture content samples were taken in each layer across the pavement cross-section. The testing was done in accordance with TMH 1 (NITRR, 1979) using Mod. AASHTO compaction where applicable.

Since the sites included cracked and uncracked surfacings, good and poor surface drainage, failed and unfailed roads, the results are applicable to most conditions. However local water-associated areas were not specifically included in the study (except in so far as they were associated with wet climates, cracked surfacings and/or poor surface drainage), and although some such water-associated areas were sampled which had free water in the pavement, the isolated occurrence of transient saturated or free water conditions lie outside the scope of this bulletin. The findings and equations in this bulletin have proved useful in investigating such moisture-associated problem areas.

This bulletin describes the moisture regime of existing pavements in South Africa, presents methods to predict moisture content in pavements, and integrates predicted moisture content with the pavement design process. The cost savings of unsoaked design are quantified.

## 2 ANALYSIS OF EQUILIBRIUM MOISTURE

Several moisture databases exist for pavements in South Africa, and these were used in this research. The two major databases used were those of Burrow and Emery. Burrow of the Transvaal Roads Department sampled subgrade materials and moisture contents in the assumed equilibrium moisture zone of pavements in the Transvaal (Burrow, 1975). The climate ranges from semi-arid to humid, and is a summer rainfall region. This data was initially analysed by Haupt of the National Institute for Transport and Road Research (1980, 1981). Emery, also of the National Institute for Transport and Road Research, sampled roads in the Cape Province and Natal (Emery, 1983a). All pavement layers were sampled in the assumed equilibrium zone, and this data was supplemented by studies of the seasonal, diurnal, and spatial variation of moisture content. The climate ranged from arid to humid, and summer, winter and all year round rainfall regions were included. This data was initially analysed by Emery (1985a).

Four other databases were occasionally used. One was a small investigation undertaken by Netterberg and Haupt (1978) of the National Institute for Transport and Road Research; it covered all pavement layers of the arid Grunau-Mariental area of Namibia and the continuously sub-humid George-Storms River area of the Cape Province. The second was from the Department of Transport (1944) in which pavement moisture contents and materials were sampled in the Transvaal and the Orange Free State. All layers were sampled in the assumed equilibrium zone, and sampling was repeated at three monthly intervals for three years. The third was a road experiment on a section of the newly constructed Kanye to Jwaneng road in Botswana. Moisture contents and suction were measured in all the pavement layers (a summary of the NITRR moisture data is in Emery, 1983b). The fourth was an investigation into basecourse distress in the Cape Province in which moisture played an important role (Sampson et al., 1985).

### 2.1 Variation Of Moisture With Climatic Area

Equilibrium moisture content results were found for each pavement layer and climatic area using data from the Emery (1983a) and the Burrow (1975) moisture studies, supplemented by data from Netterberg/Haupt (1978)(Table 1). The climatic areas were initially defined by Emery (1981) from the work by Haupt (1980) and others. The initial arid and semi-arid climatic areas were combined since the two had statistically similar results ($z = 0,14$, cf. $z = 1,96$ at 95 per cent probability); the combined area was termed arid and is defined as normal annual rainfall less than 400mm, rainfall in any season. The Cape winter rainfall area was defined as normal annual rainfall greater than

400mm, rainfall predominately in winter. The Cape continuous rainfall area was defined as rainfall greater than 600mm, no predominant wet season. The Natal area was defined as summer rainfall, Thornthwaite's Im greater than zero. The Orange Free State and the drier areas of Natal were considered to be represented adequately by the Transvaal results. The data from the Burrow study were confined to the Transvaal, and within the provincial boundary, were divided into two climatic areas: Thornthwaite's Im<0 and Im>=0. The division corresponds approximately with Weinert's N=2 contour (Schulze 1958, Weinert 1974) and fairly well with the 800mm isohyet. These two areas had statistically significantly different moisture results (z = 2,94, cf. z = 1,96 at 95 per cent probability). The delineation of climatic areas is shown in Figure 1.

The sampled pavements were over deep water tables, which is usual in South Africa (Partridge, 1967). The samples were taken 1400mm in from the edge of the travelled way, which was in the assumed equilibrium moisture zone: the validity of this assumption is proven later in this bulletin.

Table 1 Equilibrium pavement moisture contents

| Climatic area | Equilibrium moisture content (%) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Subgrade | | | Subbase | | | Base | | |
| | mean | s.d. | n | mean | s.d. | n | mean | s.d. | n |
| Arid | 6,0 | 2,8 | 131 | 5,7 | 2,9 | 19 | 3,4 | 1,5 | 26 |
| Cape (winter rain) | 6,1 | 3,6 | 81 | 6,0 | 2,7 | 17 | 4,9 | 2,0 | 16 |
| Transvaal (Im<0) | 10,3 | 5,7 | 894 | - | - | | - | - | |
| Cape (all year rain) | 10,0 | 5,3 | 98 | 5,4 | 2,0 | 20 | 3,4 | 1,2 | 19 |
| Transvaal (Im>=0) | 11,4 | 4,3 | 178 | - | - | | - | - | |
| Natal (Im>0) | 11,8 | 4,5 | 52 | - | - | | - | - | |

Note: s.d. is standard deviation

In the subgrade, moisture contents tended to increase with wetter climates. In the subbase and basecourse, no such trend was apparent. The statistical distribution of equilibrium moisture content was leptokurtic (peaked) (kurtosis = 3,23, cf. 0 for a normal distribution in the SPSS statistical package), and was skewed to the right (skewness = 1,46, cf. 0 for a normal distribution in the SPSS statistical package). The normal distribution was barely an acceptable fit to the data (chi test = 8,45, cf.

chi 0,95 = 7,81 and chi 0,975 = 9,21), and its use would give falsely higher coefficient of variation and upper confidence limits, albeit only slightly higher.

The ratio of equilibrium to Mod. AASHTO optimum moisture content (EMC/OMCm) was found for all layers and climates (Table 2). The original Transvaal results from Burrow (1975) were compacted at Proctor effort, giving a higher optimum moisture content and a lower maximum dry density than if compacted at Mod. AASHTO effort. These Proctor values were converted to Mod. AASHTO equivalents using equations derived from 784 local soils (originally discussed in Haupt, 1980):

$$OMCm = 0,80\ OMCp + 0,4 \quad \% \qquad (1)$$

with a correlation coefficient of 0,92 and a coefficient of variation of 10,7%.

$$MDDm = 0,87\ MDDp + 362 \quad kg/m^3 \qquad (2)$$

with a correlation coefficient of 0,95 and a coefficient of variation of 1,9%. These equations were newly derived and are not the mathematical inverse of those published by Haupt (1980), since inverse transformation of regression equations is statistically incorrect.

Table 2 Equilibrium to optimum moisture content ratios

| Climatic area | EMC/OMCm ratio | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Subgrade | | | Subbase | | | Base | | |
| | mean | s.d. | n | mean | s.d. | n | mean | s.d. | n |
| Arid | 0,71 | 0,34 | 131 | 0,70 | 0,26 | 19 | 0,53 | 0,24 | 26 |
| Cape (winter rain) | 0,75 | 0,45 | 81 | 0,78 | 0,28 | 17 | 0,63 | 0,16 | 16 |
| Transvaal (Im<0) | 0,94 | 0,29 | 894 | - | - | - | - | | |
| Cape (all year rain) | 0,98 | 0,31 | 98 | 0,83 | 0,28 | 20 | 0,57 | 0,17 | 19 |
| Transvaal (Im>=0) | 0,96 | 0,29 | 178 | - | - | - | - | | |
| Natal (Im>0) | 1,05 | 0,34 | 52 | - | - | - | - | | |
| Weighted mean | 0,92 | - | | 0,75 | - | | 0,58 | - | |

The statistical distribution of the EMC/OMCm ratio was normal (kurtosis = 0,2; skewness = 0,2; chi test = 0,45). It tended to increase with wetter climates in the subgrade and, to a lesser extent, in the subbase. For the basecourse it appeared independent of climate and significantly lower than for the other layers.

It is notable that the average pavement moisture content was at or below optimum moisture content for all layers and climates (except Natal subgrades), particularly since the sites included the full range of pavements, surface drainage and cracking. However within this average, some extremes may occur, such as localalised accumulations of water under loose surface seals or in cracked thick asphalt base pavements.

## 2.2 Effect Of Variables On Equilibrium Moisture Content

The effect of measured variables on equilibrium moisture contents was analysed for data of Emery (1983a) since Haupt (1980) had already analysed the Burrow (1975) data. The analysis was done using analysis of variance (ANOVA), with a breakdown into appropriate classes of the variable. Moisture contents were adjusted by covariates where possible to compensate for changes in other variables which might otherwise confound the analysis. Essentially it is used in order to correct the measured moisture contents to those which would be possessed by a standard material. The covariates most often used were plasticity index (PI), liquid limit (LL), linear shrinkage (LS), percentage of material passing the 0,425mm sieve (P425), and normal annual rainfall (MAR). Correlations between covariates and predicted dependent variable (moisture content) ranged from 0,7 to 0,9. Other covariates were tried, but were less well correlated and not used.

Climate

The effect of climate on equilibrium moisture content is shown in a breakdown by normal annual rainfall. Breakdowns were also done using Thornthwaite's moisture index, and Weinert's N-value (Emery, 1985a), and these gave similar results to rainfall.

Subgrade Subgrade moisture content was broken down by normal annual rainfall, adjusted by covariates (Table 3).

Table 3 Subgrade equilibrium moisture content, adjusted by covariates in ANOVA, broken down by normal annual rainfall

| MAR (mm) | Equilibrium moisture content (%) Adjusted mean | Actual mean | s.d. | n |
|---|---|---|---|---|
| for whole population | | 7,39 | 4,13 | 304 |
| 0 to 200 | 5,69 | 5,32 | 3,42 | 87 |
| 201 to 400 | 7,15 | 6,07 | 3,37 | 46 |
| 401 to 600 | 7,54 | 6,89 | 4,15 | 38 |
| 601 to 800 | 8,74 | 8,57 | 3,37 | 72 |
| 801 to 1000 | 9,57 | 10,65 | 4,73 | 34 |
| 1001 to 1200 | 8,78 | 13,32 | 3,59 | 6 |
| 1201 to 1400 | 10,37 | 9,40 | 4,42 | 10 |
| 1601 to 2000 | 10,42 | 8,05 | 2,34 | 11 |

Covariates used were PI, LL, P75, LS.

The variation of adjusted subgrade moisture content with normal annual rainfall was statistically highly significant (F = 14,42, df. 7,292, probability 0,000). It showed a steady increase in adjusted subgrade moisture content with increasing rainfall. The potentially confounding effect of material type on moisture can be seen by the magnitude of the adjustments made from the actual moisture contents: up to a third of the original moisture content. This highly significant relationship between rainfall and equilibrium moisture content was not identified for the Burrow data by Haupt (1980), probably because of the confounding effect of material variations.

The variation of equilibrium subgrade moisture content with climate is due firstly to a variation of soil suction with climate, fine-grained soils being more sensitive than coarse grained soils in this regard (Aitchison and Richards, 1965). Secondly it is because the suction/moisture content relationship of fine-grained materials was more sensitive to changes in suction than that of coarse-grained materials (Haupt, 1980). The subgrade equilibrium moisture content also appears to vary with climate due to an apparent predominance of coarser-grained soils in arid climates and finer-grained soils in the humid climates (Emery, 1985a). This material variation was also expected from Weinert's work (1974): where Weinert's N-value is less than 5, decomposition into fine-grained soils appears to be dominant. Where N is more than 5, disintegration appears to be dominant and coarser grained soils result.

Subbase The variation of subbase equilibrium moisture content adjusted by covariates with normal annual rainfall was not statistically significant (F = 0,68, df. 5,21, probability 0,643) (Table 4). There was no trend to change of adjusted moisture content with rainfall; the higher value in the wettest region comes from only two samples and was statistically insignificant.

Table 4 Subbase equilibrium moisture content, adjusted by covariates in ANOVA, broken down by normal annual rainfall

| MAR (mm) | n | Equilibrium moisture content (%) Adjusted mean | Actual mean | s.d. |
|---|---|---|---|---|
| for whole population | 31 | | 5,97 | 2,61 |
| 0 to 200 | 6 | 6,41 | 6,75 | 4,21 |
| 201 to 400 | 6 | 5,31 | 5,23 | 1,79 |
| 401 to 600 | 5 | 5,05 | 5,22 | 2,11 |
| 601 to 800 | 8 | 5,68 | 6,27 | 2,57 |
| 801 to 1000 | 4 | 5,66 | 5,15 | 1,97 |
| 1601 to 2000 | 2 | 8,60 | 8,10 | 1,27 |

Covariates used were PI, LL, P75, LS.

Basecourse The variation of basecourse equilibrium moisture content adjusted by covariates with normal annual rainfall was not statistically significant (F = 1,793, df. 6,20, probability 0,151) (Table 5). There was a slight trend to increasing adjusted moisture content with rainfall, but since there are so few samples in the wetter regions, it was statistically insignificant.

Table 5 Basecourse equilibrium moisture content, adjusted by covariates in ANOVA, broken down by normal annual rainfall

| MAR | Equilibrium moisture content (%) | | | |
|---|---|---|---|---|
| (mm) | Adjusted mean | Actual mean | s.d. | n |
| for whole population | | 3,89 | 1,67 | 31 |
| 0 to 200 | 3,34 | 3,77 | 1,59 | 7 |
| 201 to 400 | 3,28 | 3,37 | 1,43 | 6 |
| 401 to 600 | 4,52 | 4,02 | 0,55 | 5 |
| 601 to 800 | 3,54 | 2,94 | 1,58 | 7 |
| 801 to 1000 | 5,72 | 5,58 | 2,35 | 4 |
| 1001 to 1200 | 5,38 | 6,10 | na | 1 |
| 1601 to 2000 | 4,99 | 5,00 | na | 1 |

Covariates used were PI, LL, P75, LS.

The basecourse equilibrium moisture contents appear independent of climate because the relatively coarse basecourse materials vary little in suction with climate. They are also more closely controlled materials (relative to subgrade materials) and so their properties vary little across the country; the suction/moisture content relationship therefore varies little with climate.

Materials

A breakdown of equilibrium moisture content by material parameters was only done for subgrade materials since materials in the upper layers are relatively closely controlled and would have less variation. Covariates were not used since these were usually material parameters anyway.

Percentage passing 0,425mm sieve (P425) There was a statistically highly significant variation of subgrade equilibrium moisture content with percentage passing 0,425mm sieve (F = 7,68, df. 4,356, probability 0,000) (Table 6). The increase in moisture content as the percentage of material passing the 0,425mm sieve increased was expected. An increasingly fine-grained material should have a higher moisture content at a given suction due to the greater capillary forces in the smaller diameter pores. Over the range of suctions in southern African pavements, fine-grained materials will almost always be wetter than coarse-grained ones. A similar variation was seen with the percentage passing the 0,075mm sieve (Emery, 1985a).

Table 6 Subgrade equilibrium moisture content broken down by percentage passing 0,425mm sieve

| P425 (%) | Equilibrium moisture content (%) Mean | s.d. | Variance | n |
|---|---|---|---|---|
| for whole population | 7.94 | 4.78 | 22.88 | 361 |
| 0 to 20 | 5.42 | 2.72 | 7.43 | 15 |
| 20,1 to 40 | 6.66 | 3.17 | 10.07 | 90 |
| 40,1 to 60 | 7.32 | 3.94 | 15.53 | 92 |
| 60,1 to 80 | 8.16 | 4.81 | 23.16 | 63 |
| 80,1 to 100 | 9.87 | 6.11 | 37.35 | 101 |

Liquid limit (LL) There was a statistically highly significant variation of subgrade equilibrium moisture content with liquid limit (F = 45,08, df. 4,352, probability 0,000) (Table 7). A similar variation was seen with the other Atterberg limits.

Table 7 Subgrade equilibrium moisture content broken down by Liquid limit

| LL (%) (%) | Equilibrium moisture content Mean | s.d. | Variance | n |
|---|---|---|---|---|
| for whole population | 7,89 | 4,72 | 22,31 | 357 |
| non-plastic | 5,04 | 2,92 | 8,52 | 134 |
| 15 to 20 | 8,42 | 4,11 | 16,89 | 86 |
| 20 to 30 | 9,21 | 4,03 | 16,31 | 110 |
| 30 to 40 | 14,34 | 5,56 | 30,88 | 19 |
| 40 to 50 | 16,53 | 7,06 | 49,80 | 8 |

The strong trend for equilibrium moisture content to increase with increasing liquid limit was expected; again finer-grained materials have higher liquid limits and should have higher moisture contents than coarser-grained materials. In each group of Table 7, the average moisture content of the plastic materials was much less than the liquid limit. The subgrade equilibrium moisture content was greater than the liquid limit in only 6,4% of the plastic materials (17 out of 265 samples).

Maximum Dry Density (MDDm) There was no direct relationship between subgrade equilibrium moisture content and Mod. AASHTO maximum dry density (Table 8). However samples with densities below 1750 kg/m[3] were almost all coastal area sands which influenced the results. Re-analysis of the data excluding non-plastic materials showed a statistically significant and inverse relationship between subgrade equilibrium moisture content and maximum dry density (F = 37.08, df. 7,224, probability 0,000).

Table 8 Subgrade equilibrium moisture content broken down by Maximum Dry Density

| MDDm (kg/m[3]) | Equilibrium moisture content (%) | | | |
|---|---|---|---|---|
| | Mean | s.d. | Variance | n |
| for whole population | 7,63 | 4,62 | 21,38 | 341 |
| 1601 to 1700 | 3,51 | 2,72 | 7,40 | 13 |
| 1701 to 1800 | 8,68 | 7,97 | 63,59 | 24 |
| 1801 to 1900 | 10,45 | 7,19 | 51,68 | 43 |
| 1901 to 2000 | 9,37 | 4,06 | 16,50 | 57 |
| 2001 to 2100 | 7,62 | 3,68 | 13,58 | 64 |
| 2101 to 2200 | 6,65 | 2,39 | 5,72 | 88 |
| 2201 to 2300 | 5,65 | 2,04 | 4,17 | 51 |
| 2301 to 2400 | 3,40 | - | - | 1 |

Optimum moisture content (OMCm) The relationship between subgrade equilibrium moisture content and Mod. AASHTO optimum moisture content was statistically highly significant (F = 15,88, df. 3,305, probability 0,000) (Table 9).

Table 9 Subgrade equilibrium moisture content broken down by optimum moisture content

| OMCm (%) | Equilibrium moisture content (%) Mean | s.d. | Variance | n |
|---|---|---|---|---|
| for whole population | 7,31 | 4,15 | 17,25 | 309 |
| 0 to 5 | 2,50 | - | - | 1 |
| 5,1 to 10 | 6,48 | 3,09 | 9,59 | 222 |
| 10,1 to 15 | 9,22 | 5,39 | 29,12 | 81 |
| 15,1 to 20 | 14,32 | 6,03 | 36,41 | 5 |

# 3 PREDICTION OF MOISTURE IN THE EQUILIBRIUM ZONE OF PAVEMENTS

Prediction equations have been developed to predict moisture content in the equilibrium zone of pavements, and the ratio of equilibrium to optimum moisture content. Some prediction equations had been previously developed by Haupt (1980) from a database of plastic Transvaal subgrades. With additional data from the Cape Province and Natal (Emery, 1983a), new equations were found which predict equilibrium moisture content in all pavement layers and climatic areas of South Africa. Moisture prediction equations were also developed for expansive materials.

The prediction equations were developed using multiple linear regression (Nie et al., 1975; Hull and Nie, 1981) with a stepwise approach that allowed other variables to enter the equation if they were statistically significant. A large number of models were tried including linear, polynomial, exponential, and logarithmic. Various combinations and transformations of variables were included. The equations presented here are the best found to date.

The general prediction equations developed here apply to most gravel and soil unstabilised materials, all climatic areas and all untreated pavement layers of South Africa over a deep water table. They are valid only for pavements similar to those from which the data were obtained. This means that the design, construction, quality control, and maintenance must be similar to that in South Africa.

They only apply to pavements over deep water tables. The division between shallow and deep ranges from one metre in sands to six metres in clays (O'Reilly et al., 1968). For areas with a moisture surplus (essentially Thornthwaite's Im>0), such as most of Natal, the southern Cape, Eastern Transvaal, Cape Town, Lesotho and Western Swaziland, particular care should be taken that a shallow water table is not present, since these equations do not apply to the shallow water table situation.

The data were obtained from roads with both cracked and uncracked surfacings. The equations would apply to roads with uncracked surfacings since the moisture equilibrium is protected by the surfacing. A cracked surfacing however might destroy the assumed equilibrium moisture conditions of the upper layers. It is felt likely that the subgrade moisture equilibrium would still be protected under a cracked surfacing by the relatively low permeability basecourses and subbases typically found in South Africa. In or on the upper layers under a cracked surfacing though, localised saturation or even accumulation of water may occur, which could lead to excess pore water pressures and possible failure when repetitive loads are applied to the surface. The fieldwork here was not designed to measure transient moisture conditions during and immediately following rainfall and no account could be taken of localised saturation and ponding. An analysis of moisture contents of all pavement layers measured in various moisture studies (Emery, 1985a) showed them to be virtually unaffected by cracking of the surfacing. These were essentially long-term moisture contents and it is considered that on the sort of long-term time basis implicit in the use of unsaturated material properties in design, a cracked surfacing would not invalidate these equations in any layer. However at isolated sites with cracked surfacings where localised ponding is occurring, premature distress could occur irrespective of whether saturated or unsaturated material properties are used for design, and consideration does need to be given to the short-term effect of such transient moisture conditions.

## 3.1 Equilibrium Moisture Content Prediction Equations

General equilibrium moisture content prediction equations for all South African climates and pavement layers are presented here (from Emery, 1984). Models are presented with and without a climatic factor, and it may be seen that only a small (but statistically significant) loss of accuracy results from omitting the climatic factor. Some of the samples used in developing these models had their optimum moisture content and maximum dry density determined at Proctor compactive effort. These Proctor values were converted to Mod. AASHTO equivalents using the equations from the previous section.

The treatment of non-plastic soil Atterberg limits in the main statistical analysis was handled by developing equations without Atterberg limit terms for all materials; and equations with Atterberg limit terms for plastic materials and non-plastic materials separately. The transformed variables incorporating Atterberg limits with the percentage passing the 0,425mm sieve were developed by Haupt (1980) and could not be improved on.

All materials

Six general equilibrium moisture content prediction equations were developed from 1336 samples for use with all unbound gravel and soil materials, all pavement layers, and all climatic regions, over deep water tables:

$$EMC = 0{,}68\ (OMCm) + 0{,}031\ (LS)(P425[0{,}7]) - 1{,}1\ (K)\ \% \qquad (3)$$

with a correlation coefficient of 0,84, a coefficient of variation of 30,5%, and a standard error of estimate of 2,97.

EMC = 0,59 (OMCm) + 0,033 (LS)(P425[0,7]) + 3,72 (log{e}(100 + Im)) - 1.2 (K) - 16,1 % (4)

with a correlation coefficient of 0,86, a coefficient of variation of 29,3%, and a standard error of estimate of 2,85.

EMC = 0,054 (LS)(P425[0,7]) - 1,9 (K) + 5,0 % (5)

with a correlation coefficient of 0,81, a coefficient of variation of 33.3% and a standard error of estimate of 3,24.

EMC = 0,053 (LS)(P425[0,7]) + 4,75 (log{e}(100 + Im)) - 1,9 (K) - 16,4 % (6)

with a correlation coefficient of 0,83, a coefficient of variation of 31,4%, and a standard error of estimate of 3,05.

EMC = 1,30 (OMCm) - 1,2 (K) - 3,5 % (7)

with a correlation coefficient of 0,80, a coefficient of variation of 33.9%, and a standard error of estimate of 3,30.

EMC = 1,27 (OMCm)+ 2,73 (log{e}(100+Im)) - 1,3 (K) - 15,5 % (8)

with a correlation coefficient of 0,81, a coefficient of variation of 33.3%, and a standard error of estimate of 3,24.

These equations were developed using South African and some Namibian data. The range of material types and climates from which these data were derived is summarised in Table 10.

Table 10 Summary of climates and materials

| Parameter | minimum | mean | maximum |
|---|---|---|---|
| EMC (%) | 0,6 | 9,7 | 38,9 |
| LL (%) | 12 | 28,0 | 88 |
| PI (%) | 0 | 10,6 | 57 |
| MDDm (kg/m[3]) | 1311 | 2016 | 2396 |
| OMCm (%) | 2,5 | 10,2 | 31,2 |
| P425 (%) | 2 | 59,5 | 100 |
| P75 (%) | 1 | 31,9 | 94 |
| MAR (mm) | 110 | 662 | 2000 |
| Weinert's N | 1 | 5,1 | 50 |
| Thornthwaite's Im | -50 | -4,4 | 100 |

Notes : K = 1 for base, K = 0 for subbase and subgrade
LL Liquid limit
PI Plasticity Index
MDDm Maximum dry density at OMCm and Mod. AASHTO effort

OMCm Optimum moisture content at Mod. AASHTO effort
P425 Percentage passing 0,425 mm sieve
P75 Percentage passing 0,075 mm sieve
MAR Normal annual rainfall
Im Thornthwaite's moisture index (Im)
LS Linear shrinkage

Plastic materials

Three equilibrium moisture content prediction equations have been developed which include liquid limit and plasticity index terms. These are only for use with unbound plastic materials, but for all layers and all climatic regions over deep water tables:

$$EMC = 0{,}82\ (OMCm) + 0{,}15\ (LL[0{,}7])(P425[0{,}3]) - 0{,}8\ (K) - 3{,}2\ \% \qquad (9)$$

with a correlation coefficient of 0,85, a coefficient of variation of 30,1%, and a standard error of estimate of 2,93.

$$EMC = 0{,}78\ (OMCm) + 0{,}15\ (LL[0{,}7])(P425[0{,}3]) + 2{,}90\ (\log\{e\}(100 + Im)) - 0{,}9\ (K) - 16{,}1\ \% \qquad (10)$$

with a correlation coefficient of 0,86, a coefficient of variation of 29,4%, and a standard error of estimate of 2,86.

$$EMC = 0{,}60\ (OMCm) + 0{,}020\ (LS)(P425[0{,}7]) + 0{,}086\ (LL[0{,}7])(P425[0{,}3]) + 3.40\ (\log\{e\}(100 + Im)) - 1{,}1\ (K) - 16.1\ \% \qquad (11)$$

with a correlation coefficient of 0,86, a coefficient of variation of 28.8%, and a standard error of estimate of 2,80.

Non-plastic materials

There is no accepted enumeration of Atterberg limits for non-plastic materials in prediction equations. Here the liquid limit and plasticity index of non-plastic materials were coded to a realistic minimum value. Actual values of liquid limit reported in the data used for the study were as low as twelve. A sensitivity analysis was done on the coding value of non-plastic liquid limits by deriving moisture content prediction equations with liquid limits coded as ten, twelve, and twenty. These showed little difference in multiple regression statistics and coefficients. The equations here were derived with the liquid limits of non-plastic materials coded as twelve and the plasticity indices as zero, which were realistic approximations. These equations apply to all layers and all climatic regions, over deep water tables:

EMC = 0,55 (OMCm) + 0,25 (LL[0,7])(P425[0,3])
− 0,8 (K) − 3,7 % (12)

with a correlation coefficient of 0,85, a coefficient of variation of
30,1%, and a standard error of estimate of 2,93.

EMC = 0,49 (OMCm) + 0,26 (LL[0,7])(P425[0,3]) +
3,30 (log{e}(100 + Im)) − 0,9 (K) − 18,4 % (13)

with a correlation coefficient of 0,86, a coefficient of variation of
29,2%, and a standard error of estimate of 2,84.

EMC = 0,48 (OMCm) + 0,016 (LS)(P425[0,7]) +
0,15 (LL[0,7])(P425[0,3]) + 3,55 (log{e}(]00 + Im))
− 1,0 (K) − 17,4 % (14)

with a correlation coefficient of 0,86, a coefficient of variation of
28,9%, and a standard error of estimate of 2,81.

## 3.2 Equilibrium To Optimum Moisture Content Ratio Prediction Equations

Equations for the prediction of the ratio of equilibrium to Mod. AASHTO optimum moisture content (EMC/OMCm) are presented here. It was not possible to develop a single equation for all layers, and so equations were developed for each layer. The correlations for these equations are generally lower than those for predicting moisture content. It is likely that EMC/OMCm was more sensitive to field density than equilibrium moisture content alone, but since this was not recorded in any of the moisture studies it could not be included directly. It was included indirectly by the development of equations for separate pavement layers, although this was a less satisfactory solution and is probably responsible for the increase in unexplained variance. For non-plastic materials the Atterberg limits were coded to zero. A variety of models was tried including linear, polynomial, exponential, and logarithmic.

Subgrade

For the subgrade, equations were developed for the full range of climates and unbound materials, for unbound non-plastic materials, and for the full range of unbound materials over limited climatic ranges.

Full range of unbound materials and climates

EMC/OMCm = 0,0084 (LL[0,7])(P425[0,3]) + 0,34 (log{e}(100 + Im))

+ 0,11 (P75/OMCm)- 0,0036 (P425) - 0,89 (15)

with a correlation coefficient of 0,74, a coefficient of variation of 31,5%, and a standard error of estimate of 0,261.

Non-plastic unbound materials, all climates

EMC/OMCm = 0,19 (P75/OMCm)+ 0,0040 (Im) - 0,0036 (P425) + 0,53 (16)

with a correlation coefficient of 0,73, a coefficient of variation of 39,1%, and a standard error of estimate of 0,207.

All unbound materials, Im less than zero

EMC/OMCm = 0,13 (LS) + 0,63 (log{e}(100 + Im)) + 0,13 (P75/OMCm)
- 0,0071 (LS)(P425[0,7]) + 0,011 (PL) - 2,36 (17)

with a correlation coefficient of 0,75, a coefficient of variation of 35,0%, and a standard error of estimate of 0,243.

All unbound materials, Im greater than zero

EMC/OMCm = 0,0091 (LL[0,7])(P425[0,3]) + 0,096 (P75/OMCm)
+ 0,21 (log{e}(100 + Im)) - 0,50 (18)

with a correlation coefficient of 0,72, a coefficient of variation of 26,8%, and a standard error of estimate of 0,257.

Subbase

No satisfactory equations could be found for the prediction of EMC/OMCm for unbound materials in the subbase; the best correlation coefficient was 0,51. However in the previous section it was found that EMC/OMCm in the subbase varied little across the country, and so it could reasonably be predicted using a probability approach. The distribution of subbase EMC/OMCm was therefore described broken down by climatic area.

Im less than zero

mean EMC/OMCm = 0,707 standard deviation = 0,255 minimum = 0,258
maximum = 1,219 kurtosis = -0,642 skewness = 0,313
number of samples = 25

Im greater than zero

mean EMC/OMCm = 0,822 standard deviation = 0,286 minimum = 0,339
maximum = 1,773 kurtosis = 3,023 skewness = 1,321
number of samples = 30

Basecourse

Again no satisfactory equations could be found for the prediction of EMC/OMCm in unbound basecourses, and the same approach was used.

Im less than zero

mean EMC/OMCm = 0,560 standard deviation = 0,195 minimum = 0,220
maximum = 1.080 kurtosis = 0,526 skewness = 0,357
number of samples = 34

Im greater than zero

mean EMC/OMCm = 0,600 standard deviation = 0,173 minimum = 0,242
maximum = 0,863 kurtosis = −0,617 skewness = −0,366
number of samples = 31

Expansive Materials

One of the uses of moisture content prediction equations is in the prediction of heave of expansive materials (for example: Weston, 1980). The prediction methods lie outside the scope of this bulletin and will not be discussed in detail. However data from the moisture studies were used to develop moisture content prediction equations for expansive materials. Since the data were moisture contents from the equilibrium zone of the pavements, the equations only apply to expansive materials under covered areas where the conditions of equilibrium apply.

For this reason the dependent term of the expansive material prediction equations is shown as EMC. The equations were developed for expansive or potentially expansive subgrade materials. These are defined here by either the Kantey-Brink limits (1952): LL > 30 and PI > 12 and LS > 8 or the Wilson (1976) modified Van der Merwe limits: moderate expansion (PI*P425 12 to 23), high expansion (PI*P425 23 to 32), very high expansion (PI*P425 > 32).

Other common methods in South Africa use clay content (< 0,002 mm fraction) as a variable, but since this was not included in any of the databases, this could not be used.

(i) Kantey-Brink limits:

a) Non-expansive:

not done - use general equations

b) Expansive:

$$EMC = 0,45(LL^{0,7})(P425^{0,3}) + 5,29(\log_e(100 + Im)) - 29,50\ \% \quad (19)$$

with a correlation coefficient of 0,78 and a coefficient of variation of 23.7% and a standard error of estimate of 4,21.

(ii) Wilson modified Van der Merwe classes:

a) No expansion (PI*P425 < 12):

not done - use general equations

b) Moderate expansion (PI*P425 12 to 23):

$$EMC = 0,46\ (LL^{0,7})(P425^{0,3}) + 32,49\ (\log_e(100 + Im)) - 2,25\ (\log_e N) - 0,30\ (Im) - 152,20\ \% \quad (20)$$

with a correlation coefficient of 0,65 and coefficient of variation of 23,8% and a standard error of estimate of 3,77.

c) High expansion (PI*P425 23 to 32):

$$EMC = 0,69\ (LL^{0,7})(P425^{0,3}) + 10,96\ (\log_e(100 + Im)) - 70,49\ \% \quad (21)$$

with a correlation coefficient of 0,67 and coefficient of variation of 18,9% and a standard error of estimate of 4,11.

No equation could be presented for the very high expansion case (PI*P425 > 32) because the correlation coefficient of the best equation possible was 0,50 which was unacceptably low.

# 4 SEASONAL AND SPATIAL VARIATION OF MOISTURE

The seasonal and spatial variation of moisture content was measured in the Emery (1983a) study in climatic areas ranging from arid to humid (details of the sites are given in the Appendix). At three monthly intervals over a year, gravimetric moisture content samples were taken from each layer at various offsets across the road cross-section. The first objective was to prove that moisture contents measured in the Emery (1983a) and Burrow (1975) fieldwork were actually equilibrium values, and were not affected by abnormal rainfall or by seasonal variations. The next objective was to quantify the seasonal variation of moisture content and define the zone it occurs in, since these have considerable implications for pavement design. From this the minimum width of sealed shoulder to modify the pavement moisture regime for design was determined.

## 4.1 Validation Of Equilibrium

Normality of rainfall

To check that the rainfall during sampling was not abnormal (and in particular that sampling was not affected by drought), rainfalls during the Emery and Burrow fieldwork were compared to long term averages. As a first step, the long term rainfall patterns were examined. Tyson and Dyer (1978) had shown that extended wet and dry spells follow each other every nine to ten years in South Africa. The Emery (1983a) fieldwork was done in 1981/2 during the wet spell of 1973 to 1982; the Burrow (1975) fieldwork was done during 1973 in the same spell. Both had therefore been done in a wet spell.

However superimposed on these long term patterns, there is a variation of rainfall from year to year and place to place. Lindesay (1984) showed that spatial variations of rainfall across South Africa in a single year can be as much as -50 per cent to +200 per cent of the normal, i.e. long-term (generally 30 year) average rainfall. During the Emery fieldwork the rainfall varied from -35 per cent to +30 percent (data from the Weather Bureau monthly reports). During the Burrow study fieldwork, the rainfall varied from -25 per cent to +50 per cent of the normal over the study area (data from Lindesay, 1984). This was less than the typical variation shown by Lindesay which suggests normality of rainfall. There were also no prolonged wet or dry periods during the fieldwork.

Normality of rainfall was also tested by a comparison of actual rainfalls at the seasonal sites during 1982 with long term average rainfalls. There was no statistically significant difference between the long term average rainfall and the actual rainfall at the seasonal sites in 1982 (probability using Student's t ranged from 0,45 to 0,90, cf. 0,05 for a statistically significant difference).

The rainfalls during the Emery and Burrow fieldworks were considered normal since the variation in rainfall was within the normal range of variability, there were no prolonged wet or dry spells, and there was no statistically significant difference between actual and average rainfall at the seasonal sites.

Validation of concept of equilibrium

The pavement moisture regime was assumed to be at equilibrium under the centre of a sealed road over a deep water table, and it was assumed that a significant seasonal variation of moisture only occurred at the edges of the pavement (Figure 2). This assumption of equilibrium is important in the measurement and prediction of moisture content. There is considerable evidence (Croney, 1952; Redus, 1957; O'Reilly et al., 1968; Haliburton, 1971) that the net annual change in subgrade moisture content is negligible after two years. This equilibrium assumption was tested for South Africa.

Firstly the relationship between rainfall and pavement moisture was considered across the pavement width. The ratios of equilibrium to optimum moisture content (EMC/OMCm) at the edge (edge of the bitumen) and centreline of the pavement were plotted against rainfall in the month prior to sampling (shown in Emery, 1985a to be the best correlated rainfall) (Figure 3). There was a seasonal variation of moisture in some cases, which was more pronounced at the edge of the pavement, but the relationship between rainfall and moisture was neither clear nor direct. To clarify the relationship, plots of normalised (or z) scores were made (these being a function of mean and standard deviation for that cross-section and time) and were averaged for all the sites (Figure 4). For these cross-section drawings, the varied lane and shoulder widths at different sites were generalised into an idealised two lane road with 3,7 m lanes and one metre sealed shoulders; the generalisation did not affect the findings.

The z-scores have been ranked by rainfall to give four lines per layer: wettest month through to the driest month. If the edge was wetter than the centre during the wettest month, this would show as a positive numerically high z-score at the pavement edge for that line. Similarly in the driest month the edge should be drier and have a negative, numerically high z-score. In the basecourse there was a clear trend for the pavement edge to be wet in the wettest month. To a lesser extent the same was true in the subbase, but there was no such pattern in the subgrade. In the equilibrium zone in the central part of the pavement, there was no statistically significant trend for pavement moisture to vary directly with rainfall, supporting the concept of equilibrium. All variations in this region were less than one standard deviation ($z < 1$) and so were considered insignificant. The variations that did exist were probably due more to variations in material type than to variations in moisture.

## 4.2 Seasonal Variation Of Moisture Content

In the Emery study (1983a), gravimetric moisture content samples were taken every three months according to the pattern in Figure 5. Each season's samples were taken in undisturbed material, three metres longitudinally distant from previous samples. It was reasoned that this would be sufficient to prevent moisture ingress through previous sampling holes affecting the moisture content of new samples. It was also thought that the pavement materials would be reasonably homogeneous from hole to hole, but this proved not to be so. Unfortunately no indicator tests (such as liquid limit) were made on the moisture content samples, so it was not possible to adjust moisture contents for variations in material type between adjacent samples. The result was that there were unexpected variations in moisture from one season's hole to the next, due probably to variations in material type. This made it hard to measure the actual seasonal variation of moisture content, and use had to be made of statistical techniques to identify the edge zone of seasonal variation of moisture content.

To quantify the heterogeneity of pavement materials, an attempt was made to estimate the variation of moisture content due to material variations and test repeatability alone and not due to seasonal variations. Twenty samples were taken from one square metre of an existing sealed road. The moisture regime was normal and at moisture equilibrium. In the non-plastic unbound crushed stone basecourse, the moisture content range was 3,2% to 3,9% (mean 3,6%, standard deviation 0,2%). In the sand-clay subgrade, the moisture content range was 14,8% to 16,6% (mean 15,5%, standard deviation 0,4%). The subgrade liquid limit range was 30 to 35 (mean 32, standard deviation 1,3). The plasticity index range was 16 to 19 (mean 17, standard deviation 1,0). The linear shrinkage range was 5,9 to 8,3 (mean 7,1, standard deviation 0,6). No Atterberg Limit tests were possible on the basecourse material. These were considerable ranges within one square metre, particularly in the subgrade. An allowance was considered for repeatability errors due to testing (using the results of Smith, 1978), but there was still a considerable range unexplained and thought due to material variations. This finding supported the need for statistical techniques to identify the edge zone of seasonal variation of moisture content.

## 4.3 Edge zone of seasonal variation of moisture content

The lateral extent of the edge zone of seasonal variation has been variously reported as 750mm (Uppal, 1965), 600-900mm (Black et al, 1959), and 600-1000mm (O'Reilly et al., 1968). Because of the importance of this zone for pavement design, it was felt necessary to define its extent under South African conditions and to apply this definition to the question of a minimum width of sealed shoulder. To define the lateral extent of the edge zone of seasonal variation, the F-ratio test was used to compare the differences in variance between the edge and the equilibrium zone of the pavement. The moisture contents at each station (i.e. at each offset from the edge) were averaged over the whole year giving a mean and variance. The variance was a measure of how much the moisture content varied over the year at that station for that layer. If there had been perfect equilibrium of moisture and complete homogeneity of material, the variance should have been zero. In practice there was always a certain amount of variance due to material variations. If there was perfect equilibrium of moisture but variability of materials, the variance at each station should be the same when results across the pavement cross-section at the same site and layer were compared. If a seasonal variation of moisture content existed at the edge, the total variance at that station would be higher than the central station which only had a material variance.

The F-ratio test was applied by assuming equal variances at all stations across the pavement width, representing the variability of materials and testing. If there was no edge seasonal variation, this hypothesis would be true; if there was one, it would be false because the edge stations would vary more than the centreline ones. All the sites were at equilibrium moisture: the pavements had been constructed for greater than five years, had uncracked surfacings, fair surface drainage or better, and were over deep water tables. The average of the centreline and the inner wheeltrack station moisture contents was taken as the equilibrium condition. The F-ratio tests were made between this average and the others (Table 11). The figures underlined are statistically significant F-ratios at the 95% level. Generally the critical value was F = 2,71, df. 7,15; although this varied because the degrees of freedom varied.

Table 11 F-ratio tests to define the zone of seasonal variation of moisture content.

| Site | | F - ratio: Distance from edge of bitumen Edge | 0,6m | 1m | 1,4m |
|---|---|---|---|---|---|
| D2 | base | \3,99\ | 0,78 | 0,91 | 1,94 |
| | subbase | \8,58\ | 2,35 | 0,68 | 1,15 |
| | sel. sgde | 1,44 | 0,70 | 0,76 | 0,60 |
| | subgrade | 1,21 | 0,11 | 0,17 | 0,36 |
| CW1 | base | 2,32 | 0,74 | 0,54 | 0,34 |
| | subbase | 0,64 | 0,99 | 0,36 | 0,23 |
| | subgrade | 1,61 | 2,24 | 0,72 | 1,12 |
| G26 | base | \11,3\ | 1,85 | 2,29 | 1.09 |
| | subbase | \4,60\ | \6,24\ | 2,10 | \4,21\ |
| | subgrade | 1,03 | 2,06 | 1,04 | 1,58 |
| VD101* | base | \10,5\ | \3,88\ | 1,98 | 0,60 |
| | subbase | \3,37\ | \2,87\ | 2,69 | 1,15 |
| | subgrade | 1,99 | 1,90 | 2,16 | 0,45 |
| N2* | base | \15,9\ | 2,28 | 0,26 | 1,62 |
| | subbase | \5,98\ | 1,70 | 0,34 | 1,42 |
| | subgrade | 1,44 | \4,37\ | 2,90 | 1,41 |
| N71* | base | \12,8\ | \15,1\ | \11,2\ | 1,86 |
| | subbase | 2,61 | 2,03 | 0,73 | 1,78 |
| | subgrade | 1,67 | 1,30 | 2,61 | 0,45 |
| D20* | base | 1,77 | 1,01 | 0,63 | 0,81 |
| | subbase | 0,46 | \3,46\ | 1,37 | 2,52 |
| | subgrade | \3,01\ | \3,17\ | 0,57 | 1,44 |

*Notes. * These sites had sealed shoulders, and the nominal 1,4m station was further in from the edge of the seal. This should not affect the validity or interpretation of the results.*

Generally the variance at the edge of the surfacing and, to a lesser extent, at the 0,6 metre station was significantly different (and greater) than that of the central part of the pavement. This defined the zone of seasonal variation of moisture content as extending in from the edge of the bitumen to between 600 and 1000mm, which compares well with overseas values. There was no trend for the extent or occurrence of the zone to vary with climate or pavement type. These results also enable the edge zone of seasonal variation to be defined statistically and this is done below.

The exceptions to this were due to localised site conditions. At site N71, the road had been widened by a thick bituminous concrete surfacing, underlain only by a plastic silt-clay. This silt-clay barrier trapped water in the old inner pavement during the wet season forming a pool of free water which in turn widened the zone of seasonal variation. Since this type of construction is unusual and unsatisfactory road engineering practice, it was not considered further. The isolated differential at site G26 in the subbase at 1,4m was not supported by other readings, and was considered due to gross material variations or error.

The zone of seasonal variation appears to be more prominent in the upper layers. This was supported by Richards and Chan (1971) who predicted edge zones of moisture variation in a pavement for a range of soil profiles and climates. In an arid area (corresponding to site VD101), a zone of moisture variation was predicted at the edge in all the pavement layers. In a semi-arid area (corresponding to sites D2, D20), a zone was predicted in the upper pavement layers for all conditions, but not in the subgrade unless it was clay. In a wet area (corresponding to sites N71, CW1, G26, N2), a zone of moisture variation was again predicted in the upper pavement layers, but not in the subgrade unless it was sand or sand overlying clay.

The possibility that there was a real seasonal variation in subgrade moisture, which was being overshadowed by the relatively large material variations, was considered. A seasonal variation of heave has been found in active clays, although the moisture content of active clays may be more sensitive to change of climate than the moisture content of non-expansive materials due to differences in suction/moisture content relationship; none of these sites was over an active clay. There is conflicting evidence for seasonal variation of deflections, which would be related to a seasonal variation of subgrade moisture content, and Smith and Hewitt (1984) were unable to prove its existence. If a real seasonal variation in subgrade moisture content existed, seasonal effects should have been seen at sites with small subgrade equilibrium variance (D2 and VD101), and they were not. On balance then, a seasonal variation of subgrade moisture content was not proved.

### 4.4 Implications of edge zone of seasonal variation for design

The implications of the zone of seasonal variation of moisture on design depends on the zone position relative to the outer wheeltrack (OWT). The principle of unsoaked design is for design material properties to reflect their expected moisture condition in the pavement. If the outer wheeltrack is over the equilibrium zone (as is the case for pavements with sealed shoulders), then the moisture condition at equilibrium may be used in the design process. If the outer wheeltrack is over the zone of seasonal variation (as is the case with unsealed shoulders), then outer wheeltrack moisture conditions at the wettest time of year should be used.

The average ratio of maximum outer wheeltrack moisture content to equilibrium moisture content was found (the outer wheeltrack was defined as being 600mm from the bitumen edge) (Table 12). The ratios found were slightly higher than those reported by Bott (1980) in Western Australia, where the subgrade OWT/centreline moisture content ratio was 1,14. However they were in the same order as those used by the Main Roads Department (1981) in Queensland, where the ratio for the subgrade ranges from 1,2 to 1,4 depending on drainage and construction. The ratio of edge to equilibrium moisture content gave very similar mean values (1,47, 1,27, 1,27; for basecourse, subbase, and subgrade respectively). The average of the edge and outer wheeltrack moisture contents was used to find an edge factor for design.

Table 12 Ratio of maximum outer wheeltrack to equilibrium moisture content

| Site | Layer | | |
|---|---|---|---|
| | Base | Subbase | Subgrade |
| D20 | 1,03 | 1,01 | 1,05 |
| N71 | na | 1,01 | 1,07 |
| VD101 | 1,90 | 1,95 | 1,80 |
| G26 | 1,52 | 1,25 | 1,15 |
| D2 | 1,21 | 1,09 | 1,10 |
| CW1 | na | 1,09 | 1,10 |
| Mean | 1,41 | 1,24 | 1,21 |
| s.d. | 0,38 | 0,36 | 0,29 |

Where a pavement has unsealed shoulders, this edge factor would be used to increase the predicted equilibrium moisture content to an edge value. For the basecourse, the recommended edge factor is 1,45. For subbases and subgrades, it is 1,25. This would not apply to sites with shallow water tables since the centre of the road would be wettest there. The edge factor shows the benefit of sealed shoulders in reducing the seasonal variation of moisture content in the outer wheeltrack of the trafficked pavement.

## 4.5 Minimum Width Of Sealed Shoulders

There is a minimum width of sealed shoulder required for it to protect the travelled way which is supported by the equilibrium zone. An analysis was done to define the probability of a wheel passing over the zone of seasonal variation for various widths of sealed shoulder. The minimum width of sealed shoulder was defined as the minimum width at which this probability dropped to an acceptably low level.

The analysis calculated the joint probability of the outer wheeltrack distribution and the zone of seasonal variation distribution as the convolution of two distributions. However it was complicated further because stresses and strains induced in the pavement by the wheel load extend laterally beyond the point of loading. This "zone of influence" of the outer wheeltrack had to be found using mechanistic analysis (illustrated schematically in Figure 6).

The outer wheeltrack distribution has been well described by Kasahara (1982) who states that wheelpath distributions are generally approximated by a normal distribution. His distribution of commercial vehicle wheelpaths for a two lane, 3,7m lane width, road can be seen to be very close to normal (Figure 7), and was satisfactorily described here by a Beta distribution (the Beta distribution is described in Harr, 1977). This distribution would be expected to change with changing lane width, but his 3,7m lane is also the present standard lane width for new rural roads in South Africa and the distribution was accepted unmodified for this analysis.

The probability distributions of the zone of seasonal variation in each layer were described by Beta distributions using the data of Table 11 (Figure 8). These were considered adequate descriptions since they were based on over a thousand samples.

The zone of influence was described for a selection of pavement designs from the draft TRH 4 (NITRR, 1980) (Table 13). The four catalogue designs chosen spanned the range of materials, traffic classes, and road categories. These catalogue designs had been developed using mechanistic analysis (Maree and Sutton, 1984) with effective moduli developed partly from the HVS testing. Although the safety factor of granular materials was found using cohesion and angle of internal friction values for both the dry and wet state, often the minimum safety factor was only met in the dry state. The stresses and strains in the catalogue designs were therefore taken to represent the equilibrium moisture zone state (i.e. dry). Since design for edge zone of seasonal variation would have to be based on the wettest moisture condition, an edge zone state (i.e. wet) was defined in which the performance of the catalogue design was found using the wet state strength parameters of Paterson and Maree, 1980. The edge of the zone of inflence was defined as the offset from the load origin in the wet case at which the stresses and strains had decreased to values equivalent to the worst dry state ones. This offset was taken as the radius of the zone of influence.

The mechanistic analysis was done using the ELSYM5 programme (University of California, 1972). The standard E80 axle configuration was used with two loads at 175mm either side of the origin, each of 20 kN, and tyre pressures of 520 kPa (Paterson and Maree, 1980). The same loading configuration was used throughout this bulletin.

Table 13 TRH4 designs used for describing the zone of influence

| Construction (base/subbase) | Road category | Traffic class | Cumulative E80x10[6] | Structure |
|---|---|---|---|---|
| Bituminous/ 40AG/225BC/150G5 granular | A | E4 | 12 - 50 | |
| Cemented/ S2/150C3/300C4 cemented | B | E3 | 3 - 12 | |
| Granular/ 30AG/150G2/150C4 cemented | B | E2 | 0,8 - 3 | |
| Granular/ S2/150G4/150G5 granular | C | E2 | 0,8 - 3 | |

Material classes are as given in TRH 14 (NITRR, 1985a). S2 is a double surface treatment; AG is a gap graded asphalt; BC is an asphalt base; C3, C4 are cemented layers; G2 is a graded crushed stone base; G4, G5 are natural gravels. All these designs are underlain by a G7 granular subgrade. E80x10[6] is million equivalent 80 kN single-axle loads (ME80s).

The edge of the zone of influence was defined for each layer using the appropriate failure criteria for the material type. For granular layers, the failure criterion was a stress safety factor; for cemented and asphalt layers, it was a tensile strain limit, for subgrades it was a compressive vertical strain limit. Failure of cemented layers to an equivalent granular state was not considered in order to simplify the analysis.

As discussed above, the edge of the zone of influence was taken as the offset from the load centre (the midpoint of the two tyres) in the edge zone of seasonal variation (i.e. wet) case, at which the stresses and strains had decreased to the equivalent worst stresses and strains in the equilibrium zone (i.e. dry) case. An example is shown and for the basecourse and subgrade, the edge of the loading envelope was taken as the offset at which the strain in the wet case dropped to the maximum strain in the dry case (Figure 9). For the granular subbase, the limiting safety factor was 1,6 (Freeme, 1983), and the offset was taken where the wet case safety factor increased to that.

The edge of the zone of influence for the four pavements is described in Table 14. It can be seen that there was little variation between the pavement types, and it was taken as a radius of 250 mm in the basecourse, 300 mm in the subbase, and 350 mm in the subgrade for all pavement types.

Table 14 Edge of zone of influence of standard E80 load

| Layer | Pavement | | | |
|---|---|---|---|---|
| | C E2 | B E2 | B E3 | A E4 |
| | Edge of zone of influence (mm) | | | |
| Basecourse | 300 | 0 | 0 | 230 |
| Subbase | 300 | 310 | 250 | 370 |
| Subgrade | 370 | 310 | 400 | 430 |

To find the probability of wheel loads trafficking the zone of seasonal variation for the two lane road case, the outer wheeltrack traffic distribution, zone of seasonal moisture variation distribution, and the zone of load influence were combined. The probability of the wheel load being taken by the zone of seasonal variation was given by the joint probability of the outer wheeltrack distribution and the zone of seasonal variation distribution. The zone of influence was incorporated into this by shifting the centre of the outer wheeltrack the appropriate distance (being the radius of the zone of influence for that layer) towards the zone of seasonal variation. The basic statistics used to generate the probability distributions are given in Table 15, and the resultant joint probabilities are given in Table 16.

Table 15 Basic statistics used to generate the probability distributions of Table 16

| Layer | Basic zone of seasonal variation statistics (m) | | | |
|---|---|---|---|---|
| | mean | s.d. | minimum | maximum |
| Basecourse | 0,426 | 0,323 | 0,0 | 1,40 |
| Subbase | 0,459 | 0,308 | 0,0 | 1,40 |
| Subgrade | 0,570 | 0,315 | 0,0 | 1,40 |

| Sealed shoulder width (m) | Basic wheeltrack statistics (m) | | | |
|---|---|---|---|---|
| | mean | s.d | minimum | maximum |
| | Basecourse | | | |
| 0,0 | 0,75 | 0,258 | 0,0 | 1,55 |
| 0,6 | 1,35 | 0,258 | 0,6 | 2,15 |
| 1,0 | 1,75 | 0,258 | 1,0 | 2,55 |
| 1,4 | 2,15 | 0,258 | 1,4 | 2,95 |

| | | | | |
|---|---|---|---|---|
| | | Subbase | | |
| 0,0 | 0,70 | 0,258 | 0,0 | 1,50 |
| 0,6 | 1,30 | 0,258 | 0,6 | 2,10 |
| 1,0 | 1,70 | 0,258 | 1,0 | 2,50 |
| 1,4 | 2,10 | 0,258 | 1,4 | 2,90 |
| | | Subgrade | | |
| 0,0 | 0,63 | 0,258 | 0,0 | 1,43 |
| 0,6 | 1,23 | 0,258 | 0,6 | 2,03 |
| 1,0 | 1,63 | 0,258 | 1,0 | 2,43 |
| 1,4 | 2,03 | 0,258 | 1,4 | 2,83 |

Note: all distances assume 0,0m is the bitumen edge

It can be seen that the probability of a single wheel load being taken by the zone of seasonal variation for the two lane road case drops rapidly with increasing shoulder width, particularly past the threshold value of 0,6 metres. Compared to the unsealed shoulder case, narrow sealed shoulders of 0,6 metre reduce the probability by a factor of ten, while sealed shoulders of 1,0 metre reduce the probability by a factor of a hundred.

Table 16 Probability of a single wheel load running on the zone of seasonal variation for varying sealed shoulder widths

| Width of sealed shoulder (m) | Layer base | subbase | subgrade |
|---|---|---|---|
| 0,0 | 21,55% | 27,39% | 43,80% |
| 0,6 | 1,821% | 2,357% | 5,880% |
| 1,0 | 0,050% | 0,067% | 0,274% |
| 1,4 | 0,000% | 0,000% | 0,000% |

This was expressed as probable cumulative equivalent axles running on the zone of seasonal variation during the lifetime of the road (Tables 17 to 19), by multiplying the single axle probability with the maximum cumulative design traffic for each class. This makes the simplifying assumption that the traffic distribution of the outer wheeltrack is independent of the width of sealed shoulder. In practice there will be increased traffic on the sealed shoulder by virtue of the fact it is there. The increase is dependent on the width of sealed shoulder (wider shoulders are much more likely to attract traffic than narrow ones), provision of climbing lanes, law enforcement, and other factors which lie beyond the scope of this bulletin to evaluate.

For the heavily trafficked roads of classes E4 and E3 (design traffic greater than 3 ME80s), there was a clear advantage in shoulders wider than one metre. Simple interpolation of Table 19 shows that a 1,2 m sealed shoulder would reduce traffic borne by the subgrade (the critical case) to an arbitrary 0,05 ME80s for the E4 case. For the less heavily trafficked classes E2 and E1 (design traffic less than 3 ME80s), shoulders of one metre width are probably acceptable. For even minor roads it may be inferred that there is an advantage in providing some width of sealed shoulder, and this should also probably be one metre. The effect of sealed shoulders in reducing trafficking over the zone of seasonal variation for the two lane road case is shown in Figure 10.

Table 17 Probable cumulative equivalent axles (E80) borne by the seasonal zone in the basecourse during design life

| Sealed shoulder width (m) | Design traffic (ME80s) 50 | 12 | 3 | 0,8 |
|---|---|---|---|---|
| | Probable cumulative ME80s borne | | | |
| 0,0 | 11 | 2,6 | 0,6 | 0,2 |
| 0,6 | 0,9 | 0,2 | 0,05 | 0,01 |
| 1,0 | 0,03 | 0,006 | 0,002 | 0,001 |
| 1,4 | 0 | 0 | 0 | 0 |
| Equivalent traffic class | E4 | E3 | E2 | E1 |

Table 18 Probable cumulative equivalent axles (E80) borne by the seasonal zone in subbase during design life

| Sealed shoulder width (m) | Design traffic (ME80s) 50 | 12 | 3 | 0,8 |
|---|---|---|---|---|
| | Probable cumulative ME80s borne | | | |
| 0,0 | 14 | 3,3 | 0,8 | 0,2 |
| 0,6 | 1,2 | 0,3 | 0,07 | 0,02 |
| 1,0 | 0,03 | 0,008 | 0,002 | 0,001 |
| 1,4 | 0 | 0 | 0 | 0 |
| Equivalent traffic class | E4 | E3 | E2 | E1 |

Table 19 Probable cumulative equivalent axles (E80) borne by the seasonal zone in subgrade during design life

| Sealed shoulder width (m) | Design traffic (ME80s) 50 | 12 | 3 | 0,8 |
|---|---|---|---|---|
| | Probable cumulative ME80s borne | | | |

| | | | | |
|---|---|---|---|---|
| 0,0 | 22 | 5,3 | 1,3 | 0,4 |
| 0,6 | 2,9 | 0,7 | 0,2 | 0,05 |
| 1,0 | 0,1 | 0,03 | 0,008 | 0,002 |
| 1,4 | 0 | 0 | 0 | 0 |
| Equivalent traffic class | E4 | E3 | E2 | E1 |

# 5 PREDICTION OF EQUILIBRIUM SUCTION

Suction is an alternative measure of moisture which may be used for design on a regional basis; material strength would be found at predicted equilibrium suction and design would be done using this equilibrium strength. The use of a suction-climate relationship for predicting equilibrium suction has been established overseas (for example Aitchison and Richards, 1965), and a preliminary relationship was developed by Haupt (1980) for southern Africa. The data used by Haupt have been considerably augmented by data from Emery (1983a), and an improved suction-climate relationship is developed here for southern Africa.

## 5.1 Suction Data

Suction quantitatively describes the interaction between soil particles and water which determines the physical behaviour of the material, and it is the force which is responsible for soil-water retention. It is a pressure term (a negative one) which is a measure of the tension exerted on the water. Total potential or suction ($h_t$) may be defined in terms of energy, with components of matrix potential ($h_m$ - due to capillary forces), solute potential ($h_s$ - due to dissolved salts), gravitational potential ($z$ - due to elevational position), external pressure potential (due to variations in external pressures), and overburden potential (due to overburden) (Emery, 1985a). When considering suction in pavements, external and overburden potentials may be disregarded, and a simplified relationship used:

$$h_t = h_m + h_s + z \qquad (22)$$

Total and matrix suction are generally expressed using a logarithmic scale pF, defined as $\log_{10}h$ where h is the free energy in centimetres of water; small suctions such as those due to solute forces may also be expressed in terms of kPa. A suction of pF 3 = 100 kPa; pF 4 = 1000 kPa. Matrix and total suction measurements under covered areas had been made previously in southern Africa by De Bruijn (1963, 1965, 1973a, 1973b, 1975) and Haupt (1980). Of these data, eleven met the criteria of: measurement under a covered area, over a deep water table, and availability of original data. The measurements by De Bruijn were mainly made using gypsum blocks which measure matrix suction. There were no soluble salt measurements published, although Haupt (1980) reported an estimate that less than 0,1 per cent salts had been present in all cases. The measurements by Haupt used psychrometers which measure total suction, and soluble salt determinations were generally available. For both sets of data, actual values were used here and not the characteristic (or 85th percentile) values tabled by Haupt

(1980).

Another 27 equilibrium suction samples were taken during the Emery (1983a) fieldwork. Essentially, undisturbed suction samples were taken at a depth of 450 mm (corresponding to the eighteen inches depth used by Aitchison and Richards, 1965) in the equilibrium moisture zone of the pavement. Suctions were measured in a WESCOR psychrometric sample chamber in an environmentally controlled laboratory and soluble salt determinations were made using the NITRR saturated paste method (discussed in more detail in Emery, 1985a). The values were checked using the nomogram of Aitchison and Richards (1965) and good agreement was found.

Total and matrix suctions were calculated (Table 20) since Richards (1967) suggested that total suction gave a better correlation with Thornthwaite's Im than matrix suction, although this was barely apparent here (correlation coefficients of 0,69 and 0,68 respectively). The use of total suction enables the change in matrix suction to be examined, along with the possible effects of changes in solute suction due to leaching and so forth.

Table 20 Suction readings at sites in southern Africa

| | | | | | |
|---|---|---|---|---|---|
| Site ident- | | Source | Thornth- | Measured | Suctions |
| | Plasticity | Depth of | Calculated suctions at | | |
| ification | | | waite Im | total | solute |
| matrix | index | sample | | 450 mm (pF) | |
| | | | | (pF) | (-kPa) |
| (pF) | (%) | (mm) | total | matrix | atmospheric |
| VERY DRY | 39 | E | -50 | 3,5 | 0 |
| 3,5 | 0 | 450 | 3,5 | 3,5 | 6,0 |
| VERY DRY | 101 | E | -50 | 4,2 | 98 |
| 4,2 | 10 | 450 | 4,2 | 4,2 | 5,7 |
| VERY DRY | 4 | E | -50 | 4,0 | 280 |
| 3,9 | 12 | 450 | 4,0 | 3,9 | 5,8 |
| VERY DRY | 67 | E | -50 | 4,3 | 230 |
| 4,2 | 12 | 600 | 4,3 | 4,3 | 5,7 |
| KEETMANSHOOP | | H | -50 | 4,7 | 250 |
| 4,7 | 11 | 600 | 4,7 | 4,7 | 6,1 |
| DRY 63 | | E | -49 | 3,4 | 75 |
| 3,2 | 3 | 450 | 3,4 | 3,2 | 6,0 |
| DRY 100 | | E | -45 | 3,6 | 0 |
| 3,6 | 5 | 450 | 3,6 | 3,6 | 5,9 |
| VERY DRY | 85 | E | -45 | 3,5 | 42 |
| 3,4 | 8 | 450 | 3,5 | 3,4 | 5,7 |
| MARIENTAL | | D | -45 | 3,9 | 113 |
| 3,8 | 14 | 1700 | 4,0 | 3,9 | 6,1 |
| CAPEWINTER | 99 | E | -40 | 3,1 | 9 |
| 3,1 | 0 | 450 | 3,1 | 3,1 | 5,4 |
| VERY DRY | 15 | E | -40 | 3,7 | 40 |
| 3,7 | 4 | 450 | 3,7 | 3,7 | 5,8 |

| | | | | | |
|---|---|---|---|---|---|
| DRY 34 | | E | -40 | 3,8 | 50 |
| 3,8 | 11 | 450 | 3,8 | 3,8 | 5,8 |
| BRITSTOWN | | D | -40 | 3,4 | 44 |
| 3,3 | na | 2100 | 3,6 | 3,6 | 5,8 |
| CAPEWINTER 12 | | E | -35 | 3,3 | 77 |
| 3,1 | 0 | 450 | 3,3 | 3,1 | 5,4 |
| CAPEWINTER 26 | | E | -35 | 3,0 | 5 |
| 3,0 | 6 | 450 | 3,0 | 3,0 | 5,4 |
| DRY 38 | | E | -33 | 3,7 | 86 |
| 3,6 | 13 | 450 | 3,7 | 3,6 | 5,8 |
| CAPEWINTER 22 | | E | -30 | 3,3 | 16 |
| 3,3 | 6 | 450 | 3,3 | 3,3 | 5,4 |
| DRY 40 | | E | -30 | 3,8 | 64 |
| 3,8 | 17 | 450 | 3,8 | 3,8 | 5,8 |
| JWANENG 17 | | H | -30 | 3,6 | 44 |
| 3,5 | 0 | 500 | 3,6 | 3,5 | 5,9 |
| WELKOM | | D | -20 | 3,6 | 44 |
| 3,6 | na | 3000 | 3,8 | 3,8 | 5,8 |
| ST HELENA | | D | -20 | 3,4 | 44 |
| 3,3 | 26 | 3000 | 3,7 | 3,7 | 5,8 |
| FREDDIES SOUTH | | D | -20 | 3,0 | 44 |
| 2,7 | na | 3000 | 3,6 | 3,5 | 5,8 |
| GEORGE 120 | | E | -10 | 3,7 | 18 |
| 3,7 | 11 | 450 | 3,7 | 3,7 | 5,6 |
| ONDERSPOORT | | D | -10 | 3,7 | 29 |
| 3,7 | 35 | 3700 | 3,9 | 3,9 | 5,9 |
| CAPEWINTER 1 | | E | -5 | 3,2 | 20 |
| 3,1 | 8 | 450 | 3,2 | 3,1 | 5,4 |
| PRETORIA | | H | -5 | 4,2 | 29 |
| 4,2 | na | 375 | 4,2 | 4,2 | 5,9 |
| CAPEWINTER 102 | | E | 0 | 3,6 | 103 |
| 3,5 | 8 | 450 | 3,6 | 3,5 | 5,4 |
| VEREENIGING | | D | 0 | 3,7 | 29 |
| 3,7 | 26 | 3000 | 3,9 | 3,9 | 5,9 |
| TZANEEN | | D | 10 | 3,5 | 29 |
| 3,5 | 12 | 3000 | 3,8 | 3,7 | 5,6 |
| NATAL 71 | | E | 11 | 3,0 | 0 |
| 3,0 | 10 | 500 | 3,0 | 3,0 | 5,7 |
| CAPEWINTER 81 | | E | 20 | 3,3 | 5 |
| 3,3 | 0 | 450 | 3,3 | 3,3 | 5,4 |
| GEORGE 119 | | E | 20 | 3,3 | 9 |
| 3,3 | 0 | 450 | 3,3 | 3,3 | 5,6 |
| GEORGE 75 | | E | 20 | 3,2 | 6 |
| 3,2 | 5 | 450 | 3,2 | 3,2 | 5,6 |
| GEORGE 97 | | E | 20 | 3,6 | 9 |
| 3,6 | 5 | 450 | 3,6 | 3,6 | 5,6 |
| GEORGE 18 | | E | 20 | 3,4 | 54 |
| 3,3 | 12 | 450 | 3,4 | 3,3 | 5,6 |
| GEORGE 54 | | E | 20 | 3,4 | 18 |
| 3,4 | 13 | 450 | 3,4 | 3,4 | 5,6 |
| NATAL 2 | | E | 30 | 2,8 | 0 |
| 2,8 | 10 | 650 | 2,9 | 2,9 | 5,5 |
| MAGOEBASKLOOF | | E | 50 | 3,0 | 29 |
| 2,9 | 11 | 3000 | 3,6 | 3,5 | 5,6 |

Note that where no solute suction determinations were available, estimates were used. For sites where Thornthwaite's Im was less than −40, the estimate was 113 kPa; for sites where −40 < Im < −20, the estimate was 44 kPa; sites where Im > −20, the estimate was 29 kPa.

Reference : D De Bruijn (1963, 1965, 1973a, 1973b, 1975)
E Emery (1983a, 1985a)
H Haupt (1980)

Time of year of sampling

No adjustment of suction for the time of year of sampling was made. Suction samples taken over a deep water table below the maximum depth of seasonal variation of a uncovered area - conservatively assumed here to be three metres - had no need of adjustment, since the maximum depth of seasonal variation of moisture under these covered areas would be no greater than this. For samples taken from shallower depths no adjustment was considered necessary either because a) if there was a relationship between shallow depth suctions under a covered area and time of year of sampling, a relationship should have been found between subgrade suction (measured at 450mm below the surfacing) and rainfall, and no correlation was found (Emery, 1985a); and b) if an adjustment was needed for suctions, it should be paralleled by a adjustment in moisture content, and there was found to be no need to adjust equilibrium moisture contents for the time of year of sampling provided they were sampled in a year of normal rainfall, which was the case here.

Depth of sampling

The suction data were adjusted to a common depth, since the depth of sampling varied from 450 mm to more than 3000 mm. The suction of all unstabilised materials at a depth shallower than the depth of seasonal variation was found from the equation (Richards, 1967):

$$(h_m)_z = (h_m)_{z_o} + z_o - z \qquad (23)$$

where $(h_m)_z$ is the matrix suction in cm. at the depth z cm.
$(h_m)_{z_o}$ is the matrix suction in cm. at a depth $z_o$ cm.

Solute suction

For some of the De Bruijn and Haupt measurements no solute suction determinations were available. For measurements made with gypsum blocks the blocks provided a degree of buffering and the need for solute suction determinations was reduced, but for measurements made with psychrometers, there was no buffering and solute suction determinations were needed.

These were estimated using averages of the solute suctions determined by Emery (1983a, 1985a), since the variation of solute suction across southern Africa appeared to be small, and primarily related to climate (doubtless partially a function of degree of saturation). For arid climatic areas (Thornthwaite's Im < -40), 113 kPa was used; for semi-arid areas (-40< Im < -20), 44 kPa; for sub-humid and wetter areas (Im > -20), 29 kPa.

Atmospheric humidity

The equivalent suction due to atmospheric humidity ($h_a$) was found for each site using atmospheric relative humidity (RH) in the equation from Richards (1965):

$$h_a = 6{,}502 + \log_{10}(2 - \log_{10}RH) \quad pF \qquad (24)$$

This equation assumes a temperature of 20°C, which was close to the actual air temperatures and no adjustment for temperature was made; the error due to this was considered insignificant. The mean atmospheric relative humidity was found by averaging the 08h00 and 14h00 average readings for the month of sampling at each site. The relative humidity data for southern Africa were limited and weather stations were not always close to the relevant site. The records themselves were of short duration and not always accurate (Schulze, 1980). The resultant equivalent suctions were therefore somewhat suspect, but at least were all of the same order of magnitude.

Overseas researchers (Russam, 1962; Richards, 1967) had found covered sites where the soil suction was equal to the equivalent atmospheric humidity suction. Richards described a site at Woomera where total suction was controlled by mean atmospheric humidity. The matrix suction was approximately zero, and total suction was due to solute suction alone. Under covered areas in southern Africa though, either the rainfall is too high, the soluble salts too low, or the water table is not deep enough, for this to occur. It was felt unlikely that the limited operating range of the psychrometers (maximum suction about pF 4,5) meant that very high suctions were not recorded, since suctions below pF 4,5 were recorded at sites in the arid regions. In addition no sites were found where the vegetation in the adjacent veld had wilted (the wilting point of indigenous vegetation is less than pF 4,5).

## 5.2 Climate Data

The suction-climate relationship is generally based on Thornthwaite's moisture index (Im), which is a dimensionless index of rainfall and evapo-transpiration (Thornthwaite, 1948). Earlier research (such as Aitchison and Richards, 1965) had shown this to be the best measure of climate in the suction-climate relationship. This was tested and it was confirmed that it was better correlated with total suction (r = -0,72) than Weinert's N-value (Weinert, 1974) (r = 0,31) or normal annual rainfall (r = -0,64).

The first map of Thornthwaite's Moisture Index (Im) for South Africa was drawn by Schulze (1958). This was based on over 400 stations in South Africa, Lesotho, South West Africa, Botswana, and Swaziland. However only a small size copy of this map was available with limited contours and this was inadequate. A new map of Thornthwaite's moisture index was prepared to a much larger scale (A0), using the original data of Schulze and supplementing these by Indices calculated at another hundred stations in Mozambique, Botswana, South West Africa, and South Africa (data from Thornthwaite Associates, 1962). These new stations improved contour accuracy at the borders of South Africa (where original data were few) and gave better contour resolution in the very wet areas of eastern South Africa.

The map was drawn using a computer generated model and a computer plotting package (Control Data, 1983). The mathematical model was a grid of 135 000 points overlying southern Africa, and was constructed using 504 Moisture Indices (Im). The moisture indices at the grid intersections were found by linear interpolation from five points with no cutoff, and these were contoured at intervals of Im = 10.

In areas of steep moisture index gradient such as the eastern Transvaal and Evelyn Valley near King Williams Town, the contours were too closely spaced to be usable, and were simplified. In isolated mountain areas such as the eastern highlands of Zimbabwe, there were insufficient meteorological stations to define the limits of the wet and dry areas accurately, and interpolation was needed using rainfall and topographical maps to define the limits of the wet areas. There are slight differences between the Schulze map and this new one, mainly in the very wet and very dry areas.

A reduced size copy of the full map is shown to illustrate its scope (Figure 11), and a simplified A4 version is shown to use as an approximate guide (Figure 12). A full sized copy of the new map is available from NITRR (drawing 940-0-4184a).

### 5.3 Suction/Climate Relationship

To develop a new southern African suction-climate relationship, the field total suction data (at a depth of 450 mm below the surfacing) from Table 20 have been against Thornthwaite's Im (Figure 13). As a comparison the total suction line used for design purposes of Aitchison and Richards (1965) and the heavy clay matrix suction line from Russam and Coleman (1961) are included in the figure. These heavy clays were generally at high suctions, and their total suction should be similar to the matrix suction (because the logarithmic nature of suction means that solute suctions only really enter the total-matrix suction relationship at low total-matrix suctions.

The degree of change of suction with climate varies with material type. The low plasticity southern African materials (defined as plasticity index less than three which was a natural grouping of the data available) varied little with climate, and the same was seen for the sands of Russam and Coleman. The local suctions were higher overall than those of Russam and Coleman, due probably to the effect of solute suction. It was felt unlikely that the limited operating range of the psychrometer (lower limit pF 2,8) meant that very low suctions were not recorded, because suction measurements were confined to covered area sites with deep water tables; the psychrometer lower limit corresponds to a water table depth of 6,7m and over most of South Africa the water table is deeper than that.

The more plastic southern African materials (defined as plasticity index greater than or equal to ten which was a natural grouping of the data) showed a variation of suction with climate similar to the data of Russam and Coleman, and Aitchison and Richards. The Australian materials of Aitchison and Richards were generally at higher suctions in the arid regions due to either differences in soluble salts and/or plasticity (Table 21) or to differences in test methods (Aitchison and Richards estimated total soluble salts using the electrical conductivity method of Piper (1950); the southern African soluble salts were generally found using the NITRR saturated paste method described in more detail in Emery, 1985a). No solute suction data were available from Russam and Coleman, but the materials were classified as heavy clays which were probably considerably more plastic than the southern African materials sampled here.

Table 21 Comparison of southern African suction samples with those of Aitchison and Richards (1965)

| | Soluble salts (%) | | | Plasticity Index | |
|---|---|---|---|---|---|
| | mean | range | n | mean | n |
| Aitchison and Richards | 0,36 | 0 to >2,0 | 30 | 45,0 | 12 |
| Southern Africa | 0,07 | 0 to 0,30 | 16 | 7,4 | 16 |

In the wet regions, the southern African materials tended to be drier than the others, due probably to a lower incidence of shallow or perched water tables. At many Australian sites temporary perched water tables were a distinct possibility due to the plastic material, and at the sites where Im > -10 the profile drainage was restricted or worse. This may also have been the case for the Russam and Coleman sites; although no water tables were found at their sites, the combination of heavy clays and wet climates suggests that perched water tables were likely. In contrast the southern African sites were over deep water tables and with the relatively less plastic insitu materials, perched water tables were less likely.

## 5.4 Suction In Design

An approach to design was proposed by Aitchison and Richards (1965) who used the suction-climate relationship to find suction values on a regional basis. Haupt (1980) modified this to include a characteristic value of suction; defined as a suction with only a limited probability of being exceeded. The suction-climate relationship was more suited to this than moisture content prediction equations because it enabled minimum probable suction to be presented on a regional basis. Here, minimum probable suction was found by deriving the equation of the southern African suction-climate relationship in Figure 13, and applying an appropriate confidence interval to it. Since there were fewer than 30 samples, the standard error of the mean was significant, and the t-statistic (rather than the z-statistic) was necessary. The confidence interval (Younger, 1979) was given by:

$$\text{Confidence interval} = h_{p} \pm t.(SEE).(A) \qquad (25)$$

where $h_{predicted}$ = predicted suction from the equation
- t = Student's t (16 d.f., one tail, 1,06 at 85%, 1,35 at 90%, 1,76 at 95%)
- SEE = standard error of estimate
- $A = (1 + 1/n + (Im - Im_{mean})^{2}/SXX)^{0,5}$
- n = number of samples
- Im = Thornthwaite moisture index
- $Im_{mean}$ = mean value of Thornthwaite moisture index
- SXX = corrected sum of squares

There were too few data to test for homoscedasticity, i.e. the scatter of points about the line is the same for every X-value (Younger, 1979); but Figure 13 shows no trend for changing scatter of points with X-value and so homoscedasticity was assumed. A satisfactory equation could only be derived for the suction-climate relationship of plastic materials (PI greater than ten), even though a variety of models was tried including linear, exponential, logarithmic, and polynomial. The equation of statistical best fit to predict total suction at a depth of 450mm below the surfacing ($h_{t450}$) under a covered area was:

$$h_{t450} = -0,010\ (Im) + 3,63 \ \text{pF} \qquad (26)$$

with a correlation coefficient of 0,75, a standard error of estimate of 0,295, and a coefficient of variation of 7,8%.

Richards (1967) had suggested that sites with poor drainage should have their Thornthwaite moisture index increased to allow for the extra water present. The Thornthwaite Im of sites with poor surface drainage was increased by ten which was felt to be a reasonable first estimate and new models tried; these had correlations between suction and climate similar to the original. In the absence of data on the true magnitude of the Im increase however, it could only be an arbitrary one and was not pursued.

Although no statistically valid equation could be found for the suction-climate relationship of slightly plastic materials, an examination of the data suggests that minimum probable suctions (at an 85 per cent probability) of pF 3,1 for climates where Im < 0, and pF 3,0 for climates where Im > 0 would be appropriate, and this could be further simplified to pF 3,0 for all climates for practical purposes. For materials with plasticity indices between three and ten, the minimum probable suction should be found by interpolation.

The minimum total suction (at a depth of 450 mm below the road surface) for southern Africa at an 85 per cent probability is shown in Figure 14. The proposed local design curve of Haupt (1980) is also shown. At Thornthwaite's Im = 60, the minimum probable suction is pF 2,6, while Haupt's local design curve gives a suction of pF 1,5. Assuming no solute suction, these are equivalent to water table depths of 4,0 m and 0,3 m respectively. Since these models apply to sites with deep water tables, a depth of 4,0 m is felt more likely, and Haupt's design curve value is rejected. For Thornthwaite's Im = -60, the minimum probable suction is pF 3,9, while Haupt's local design curve gives a suction of pF 5,1; the former is considered more reasonable in the light of the measured values and experience, and again Haupt's design value is rejected.

## 5.5 Map of minimum probable suction for design in southern Africa

A map of minimum probable suction was prepared using the suction-climate relationship at the 85 per cent confidence limit for materials with a plasticity index of ten or more, and the new Thornthwaite's Moisture Index map (Figure 15). It gives a minimum probable total suction at a depth of 450 mm below the surfacing of a covered area on a regional basis.

## 5.6 Limitations of suction for pavement design in South Africa.

The value of suction for pavement design in South Africa at present probably lies in some application to regional design in the planning stages of a project. The significant advances made in the use of moisture content in pavement design (described in the following sections) coupled with the practical difficulties of suction measurement and instrumentation, mean that the use of suction for detailed pavement design will be limited in southern Africa; moisture content will be the measurement of material wetness for some time to come. The overseas use of suction for estimating material wetness in the shallow water table situation (one such approach is described in Road Note 31, 1977) is unlikely to be adopted in South Africa in the near future due to uncertainties about the seasonal variation of water table and the effect of unusually wet periods.

# 6 USE OF PREDICTED MOISTURE CONTENT FOR DESIGN PURPOSES

The potential uses of predicted moisture content in design are many; for example it has application in modifying existing pavement designs and/or material standards, it may be used to develop new and more economical pavement designs, or it may be used to more accurately predict the swell of expansive materials. Each application will require significant research effort to develop and test, and so work to date has concentrated on the establishing the methodology and techniques common to each application. This was done using the application of modifying an existing pavement design method and material classification (TRH 4 and TRH 14; NITRR, 1985b and 1985a respectively), and this is described here.

In South Africa, present pavement design methods include soaked CBR for classification of untreated materials (TRH 14, NITRR, 1985a). The assumption is made that the materials might reach a soaked condition in practice. However with the ability to predict equilibrium (or field) moisture within the road profile, it is now possible to design using field (or unsoaked) CBR where appropriate. In the past the objections to this approach have been that it increased the risk of failure, and there are a lack of suitable test methods for unsoaked CBR determination. In this bulletin, this risk is quantified for five existing designs, and the cost of using an unsoaked approach is compared with the cost of the soaked approach; a modified CBR test method is proposed. The methodology developed here will also serve as the model for extending the use of predicted moisture content to other applications.

Considerable work has been done in South Africa by NITRR on pavement design, and a catalogue of designs has been developed as part of TRH 4 (NITRR, 1985b). These designs were prepared using mechanistic analyses and assumed certain moduli for each layer. The subgrade modulus was assumed to be that of the wet state, and the subbase and basecourse moduli were assumed to be those of a drier state (Freeme, 1983; Maree and Sutton, 1984). Since the present study has shown that subgrades are often not in the wet state, there is the potential to use unsoaked material strengths in the subgrade. This approach was also suggested by Maree (1982) who did much of the development work for TRH 4, and is again suggested in TRH 4 (NITRR, 1985b). The cost savings of this new unsoaked approach in TRH 4 will be fairly small because TRH 4 already incorporates a form of unsoaked design in the upper layers, and so savings have already been made. However the methodology and techniques will serve as a guide to incorporating unsoaked design into other pavement design methods where greater savings can be made in the upper pavement layers.

To quantify this potential, the risks and costs of using unsoaked subgrade strengths were found within the framework of the existing TRH 4 design method. The minimum subgrade modulus needed for a pavement to carry its design traffic was estimated. A low quality subgrade material (on the basis of soaked CBR) was chosen, and its equilibrium moisture content predicted. Since modulus of an unsoaked material is generally greater than that of a soaked material (depending on the type of test and conditions of drainage), this lower quality subgrade was chosen so that its modulus at its equilibrium moisture content was equal to the minimum modulus required. The risk of failure was defined as the probability that the actual equilibrium moisture would be greater than predicted; thus the modulus would be less than assumed and less than the minimum required; and the pavement would reach terminal condition prematurely. This could require premature rehabilitation and would increase rehabilitation costs.

## 6.1 Variation Of Strength With Moisture

That there is a variation of gravel and soil material strength with moisture is well known. However this variation had not previously been adequately quantified for either modulus or California Bearing Ratio (CBR), and this had to be investigated first.

Variation of modulus with moisture

A variation of modulus of gravel and soil materials with moisture is known (for example Monismith et al., 1971, Chou, 1977). However the available literature is undefined as to the exact form of the variation and a number of models to predict modulus from moisture content and suction had been developed with varying success.

Seed, Chan and Lee (1962) investigated the resilient characteristics of the AASHO road test subgrade soil and found that resilient modulus was extremely sensitive to moisture content above the optimum moisture content. A large variation of resilient modulus with moisture content was also shown by Mitry (1965), who found the resilient modulus of an uncovered subgrade to be 75 per cent greater at the end of the dry season than after rain.

Visser (1981) developed models to predict subgrade resilient modulus using data from Brazil. He was unable to include moisture into the model in a satisfactory manner and concluded that his data set was probably deficient in its ability to predict moisture content influences, possibly as a result of laterisation (Visser et al, 1983). His short model would not fit U.S. data until modified by the inclusion of moisture content (unpublished, but later analysed and published by Austin Research Engineers, 1983, who developed models to predict subgrade resilient modulus from several variables, including moisture content. An ARE relationship between resilient modulus and suction for till showed a very substantial increase in resilient modulus as the material dried from the saturated state to optimum moisture content). A similar pattern was shown by the variation of resilient modulus with degree of saturation for granular materials (Rada and Witczak, 1981), although the exact relationship depended on aggregate type, with natural gravels being more sensitive than crushed stone.

The consensus of the above results is that modulus is a function of moisture state and of material type (it is also a function of load, but this is outside the scope of this bulletin). There is a large increase in modulus as a material dries from saturation to the optimum moisture content, and this increase is proportionally greater for lower quality materials under the pavement loading conditions. However given the complex nature of modulus, at present and with the data available, the most appropriate method of predicting the variation of modulus with moisture content is considered to be indirect, using a modulus-CBR and a CBR-moisture content relationships.

## 6.2 Prediction of modulus from CBR

Probably the most common method of predicting modulus (E) from CBR is by equations of the form E = k (CBR), where k is a constant of proportionality. In the past, a value of 10,4 has often been taken for the constant, but this can vary substantially. Visser et al (1983) mentioned constants ranging from 1,7 to 79,0 for soils not subject to laterisation. Both moisture content and density were shown to affect the relation between modulus and CBR. Despite this variation though, such equations still remain the best way of predicting modulus from CBR. Austin Research Engineers (1983) developed an extensive database from published modulus results. They concluded that relationships exist between subgrade modulus and empirical soil strength tests (such as CBR), but the development of such relationships was not cost effective within the constraints of the research they were commissioned to do.

However an improved relationship was given by Paterson and Maree (1980) who used a varying constant of proportionality. This same approach was used here, based on the data of Freeme (1983) who gave effective moduli for different granular materials in the wet and dry states. The foundation of the values of many of these has been determined from the wide variety of Heavy Vehicle Simulator tests carried out on South African roads in the past few years. From these, and the soaked CBR (CBRs) criteria for each material class (TRH 14: NITRR, 1985a), the constant of proportionality for materials in the soaked state was found over the CBR range from CBR 3 to CBR 80:

$$k = 30,79\ (CBRs)^{-0,558} \qquad (27)$$

with a correlation coefficient of 0,99 (a standard error of estimate is not reported here because the relationship is not linear and the error term of the transformed equation would vary across the range of the independent term). After incorporating equation (27) into the E = k (CBR) relationship, it compares well with the relationship of Powell et al. (1984) over the range of CBR 2 to 12 (Figure 16):

$$E = 17,6\ (CBRs)^{0,64} \qquad (28)$$

and led to an equation relating modulus to soaked CBR for South African materials over the CBR range 3 to 80:

$$E = 30,79\ (CBRs)^{0,44} \qquad (29)$$

with a correlation coefficient of 0,99 (a standard error of estimate is not reported for the reasons discussed above).

Variation of CBR with moisture and density

Both moisture content and density were known to affect the modulus-CBR relationship and the effect of both had to be taken into account in predicting modulus. This was done here by developing a relationship between CBR, moisture content and density, and using that to modify the modulus-CBR relationship. There were considerable data available on field dynamic cone penetrometer CBR, laboratory soaked CBR, and field moisture content (Burrow, 1975). These were supplemented by a laboratory study on the variation of CBR with moisture content (Shackleton and Emery, 1985).

In the laboratory study, six different granular materials ranging from material classes G3 to G10 (TRH 14: NITRR, 1985a) were all compacted at Mod. AASHTO optimum moisture content and an appropriate compactive effort. For material classes G3 to G5 (usually basecourse and subbase), the compactive effort was Mod. AASHTO. For material classes G6 to G10 (usually subgrade) the compactive effort was Proctor. This was done so that the laboratory compaction reasonably replicated the compaction of pavement layers. For each material, one sample was tested after four days soaking, one at optimum moisture content, one was dried back for four days, and one was dried back for seven days (drying at 50°C, equilibrated at room temperature for up to 48 hours in a plastic bag after drying). The moisture contents ranged from about one quarter optimum to soaked. The sample preparation was chosen to simulate the processes involved in pavement construction and the subsequent wetting/drying, since the stress-strain path during preparation affects the CBR result (Turnbull and McRae, 1952).

The weakest materials in the soaked state showed proportionally the greatest increase in strength as they dried out, but in all cases there was an increase in strength at optimum moisture content compared to soaked moisture content. Relationships were found between soaked CBR (CBRs) and unsoaked CBR (CBRu) at various ratios of actual moisture content to Mod. AASHTO optimum moisture content (mc/OMCm):

for mc/OMCm = 1,0 CBRu = 11,02 (CBRs)[0,52] (30)
for mc/OMCm = 0,75 CBRu = 20,88 (CBRs)[0,45] (31)
for mc/OMCm = 0,5 CBRu = 46,53 (CBRs)[0,33] (32)

with correlation coefficients of 0,92, 0,89, and 0,96 respectively (again a standard error of estimate would be misleading because of the non-linear nature of these equations). From these, the ratio of unsoaked to soaked CBR was found for materials classes G4 to G10 (Table 22). The change in CBR with moisture content paralleled the change in modulus with moisture content discussed above.

Table 22 Variation of CBR with moisture content

| Material class (TRH 14) | Soaked CBR (%) | mc/OMCm ratio 1,0 | 0,75 | 0,5 |
|---|---|---|---|---|
| | | Ratio of unsoaked to soaked CBR | | |
| G4 | 80 | 1,34 | 1,88 | 2,47 |
| G5 | 45 | 1,77 | 2,57 | 3,63 |
| G6 | 25 | 2,35 | 3,56 | 5,38 |
| G7 | 15 | 3,00 | 4,71 | 7,58 |
| G8 | 10 | 3,65 | 5,88 | 9,95 |
| G9 | 7 | 4,33 | 7,16 | 12,63 |
| G10 | 3 | 6,50 | 11,41 | 22,29 |

A complex model was then developed to describe the complete relationship between soaked and unsoaked CBR over the range of moistures. A great variety of simple relationships were tried including linear, polynomial, exponential, and logarithmic, but none gave good correlations. Eventually the form of the model was derived from first principles and repeated regression runs were made to establish the coefficients. For the laboratory data:

CBRu = 56,54 (e[-1,42 mc/OMCm])(CBRs[0,48]) % (33)

with a correlation coefficient of 0,94.

CBRu = CBR of a specimen at any moisture content less than saturation
moisture content (shown here as mc); with the specimen compacted at Mod. AASHTO OMC; and with Mod. AASHTO compactive effort for basecourse and subbase quality material and Proctor compactive effort for subgrade quality material.
mc = actual moisture content
OMCm = Mod. AASHTO optimum moisture content
CBRs = soaked CBR

A similar model was developed from the data of the Burrow (1975) study. These data included field dynamic cone penetrometer results, field moisture contents, Proctor optimum moisture contents, and laboratory Proctor soaked CBRs. The equivalent field CBR was found from the DCP results using the equation:

$$\log_{e}CBR = -1{,}30 \log_{e}DCP + 6{,}31 \quad (34)$$

from Kleyn (1974). Since Burrow's results were at Proctor optimum moisture content, these were converted to equivalent Mod. AASHTO values using equation (1). No field densities were recorded, and so it was assumed to be equivalent to that at Proctor compaction since all the samples were from the subgrade. There were 245 sets of data which were stratified into 33 classes.

From this data, a relationship was found between field CBR at field moisture content, laboratory soaked CBR, and the ratio of actual or field moisture content to Mod. AASHTO optimum moisture content:

$$CBRu = 53{,}52 \left(e^{-1{,}12\, mc/OMCm}\right)\left(CBRs^{0{,}45}\right) \% \quad (35)$$

with a correlation coefficient of 0,96. Since this equation was very similar to equation (33), the two data sets were merged and a general equation developed:

$$CBRu = 59{,}13 \left(e^{-1{,}33\, mc/OMCm}\right)\left(CBRs^{0{,}46}\right) \% \quad (36)$$

with a correlation coefficient of 0,95. This was taken as the best model relating soaked and field CBR.

Prediction of modulus from CBR and moisture

The modulus/soaked CBR relationship was established using a varying constant of proportionality (equation 27). The complementary unsoaked/soaked CBR relationship was also found (see equation 36). These two equations were integrated to give a single equation which predicts modulus (E) at any moisture content (less than saturation moisture content) from laboratory soaked CBR:

$$E = 187\,(e^{-0,59\,mc/OMCm})(CBR_{s}^{0,20})\ \text{MPa} \qquad (37)$$

The correlation coefficient was 0,95, estimated as a multiple coefficient of determination (Younger, 1979) from the sum of squares of the two constituent equations.

For the granular material classes G3 to G10 of TRH 14 (NITRR, 1985a), moduli were predicted for varying moisture contents (Figure 17). The practical upper limit to moisture is the soaked condition. For each material class, the ratio of soaked to optimum moisture content was found, and its 85th percentile value plotted on Figure 17 to delineate the probably soaked area. There was good agreement between this and values that would be given by equation (37). The range of moisture contents typically found in South African pavements is highlighted on Figure 17, and these were taken as plus/minus one standard deviation about the average EMC/OMCm ratios for different climatic areas.

## 6.3 Application Of Predicted Moisture Content: Trh 4 Design Using Unsoaked Subgrade Strength

The first application of predicted moisture content was to the subgrades in TRH 4 design method (NITRR, 1985b). This, in common with almost all road design in South Africa, is done on the basis of soaked subgrade strengths. The TRH 4 design method also assumes that all subgrades are brought up to a uniform support standard. Subgrade materials are classified by soaked CBR and, to a less important extent, by Atterberg limits. They are divided into four strength groups and the subgrade preparation is specified to give a soaked CBR greater than 15 in the upper subgrade layer (Table 23).

Table 23 Preparation of subgrade (TRH 4)

| Design CBR of subgrade (%) | <3 | 3 - 7 | 7 - 15 | >15 |
|---|---|---|---|---|
| Add layers | n.a. | 150mm G7, 150mm G9 | 150mm G7 | - |
| Insitu treatment | | rip and recompact | | |

Proposed subgrade strength criteria

It is proposed that subgrade strength criteria be based on both soaked CBR and optimum moisture content CBR (CBRomc). An optimum moisture content CBR test would be done in which material is prepared and compacted in the same way as the existing soaked CBR test (TMH1: NITRR, 1979), but the sample tested immediately at optimum moisture content. Materials with a soaked CBR greater than 15 would be acceptable as the upper subgrade layer (selected layer); and materials with a soaked CBR less than 15 but a CBRomc greater than 15, may be acceptable as the upper subgrade layer if the risks of failure are sufficiently low. Thus a granular material which met the Atterberg Limit requirements of the G7 class, but had a soaked CBR of less than 15, may be accepted under the proposed unsoaked criteria as equivalent to a G7 (G7E) if the CBRomc was greater than 15. Materials with a CBRomc less than 15 will not be acceptable as the upper subgrade layer.

## 6.4 Risk Of Premature Terminal Condition Using Unsoaked Design

For subgrades with a soaked CBR greater than 15, the risk of premature terminal condition when using the proposed unsoaked strength criteria remains unchanged from the present and will not be considered here. For subgrades with a soaked CBR less than 15, the risk is increased and is the probability that the field subgrade modulus will be insufficient to carry the design traffic. This risk was found for five representative catalogue designs, chosen to span the range of traffic, road categories, and pavement types used in South Africa (Table 24).

Table 24 TRH 4 catalogue designs used for analyses

| Construction (base/subbase) | Road category | Traffic class | Cumulative ME80's | Structure |
|---|---|---|---|---|
| Bituminous/ granular | A | E4 | 12 - 50 | 40AG/225BC/150G5 |
| Cemented/ cemented | B | E3 | 3 - 12 | S2/150C3/300C4 |
| Granular/ cemented | B | E2 | 0,8 - 3 | S2/125G2/150C4 |
| Granular/ granular | C | E1 | 0,2-0,8 | S2/100G4/150G5 |
| Granular/ granular | C | E0 | < 0,2 | S2/100G4/125G5 |

where ME80 is million equivalent 80 kN single-axle loads, and the material classes are given in TRH 14 (NITRR, 1985a) and TRH 4 (NITRR, 1985b).

For each design, mechanistic analyses were used to quantify the modulus/traffic relationship for varying subgrade moduli (Figure 18). A reliability function for this relationship has been incorporated by the use of road category. The allowable safety factors for granular layers (Maree, 1978), and the allowable strains for bound layers and subgrades were a function of road category (Freeme, 1983) as was the possible terminal condition of the road. Road category was assumed to be an adequate expression of the uncertainty associated with mechanistic design, and the relationship between modulus and traffic was taken as deterministic. At the current state-of-the-art, this is probably the best allowance that can be made.

The modulus of lower quality subgrades can be expressed as a probabilistic function of CBR and moisture: neither can be explicitly defined. There is uncertainty associated with both of these (mathematically defined by individual probability density functions), and the uncertainty associated with modulus is expressed as a joint probability density function of the individual probabilities. However there is now sufficient data available to model these individual probability density functions, and to find the probability function of field modulus.

Individual probability density functions were developed for the soaked CBR of lower quality subgrades and for the predicted ratio of equilibrium to optimum moisture content. For soaked CBR, the probability function was defined in two ways. Firstly, using a database developed from 307 old laboratory records (Haupt, 1980), the distribution of soaked CBR for samples with a soaked CBR between 7 and 15, and a CBRomc greater than 15, was found. It was normally distributed (mean 11,7, standard deviation 2,0, skewness -0,645, kurtosis -0,204, number of samples 23). The soaked CBR distribution for samples with a soaked CBR less than 7 and a CBRomc greater than 15, could not be defined as only two samples met this criteria; the use of materials with such a large difference between soaked and CBRomc in the upper subgrade layer was also considered to be risky at this initial stage of unsoaked strength design, and so was not pursued. Instead it was felt that such materials should be overlaid with a selected layer which itself could be judged using the proposed unsoaked strength criterion. The analysis therefore concentrated on the G7E lower quality subgrade (7<CBRs<15 and CBRomc>15).

Secondly the distribution of soaked CBR for the G7E material was assumed to be uniform, i.e. there was an equal probability of the soaked CBR being any value between 7 and 15 (mean 11, standard deviation 2,31). This distribution was similar to that found from the laboratory data and either could have been adopted. In the event the uniform distribution was taken to define the probability density function of soaked CBR for G7E lower quality subgrades.

The probability density function for predicted ratio of equilibrium to optimum moisture content was derived from the prediction equations presented earlier. The equation for the full range of materials and climates (equation 15) had a standard error of estimate of 0,261. It was shown earlier that the ratio of equilibrium to optimum moisture content was normally distributed, and it was shown elsewhere (Emery, 1985a) that homoscedasticity may be assumed for the predicted ratio of equilibrium to optimum moisture content. The probability density function of the predicted ratio of equilibrium to optimum moisture content was therefore taken as normally distributed with the mean being the predicted value, and a standard deviation of 0,261.

The probability density function of modulus was given by the individual probability density functions of soaked CBR of G7E materials and the ratio of equilibrium to optimum moisture content. This joint probability density function was found for various moistures using the point estimate method (McGuffey et al., 1981; Grivas, 1984) (Table 25). Here Y = fn(X1, X2) where Y was modulus, X1 was the ratio of equilibrium to optimum moisture content (EMC/OMCm), and X2 was soaked CBR (CBRs). The correlation between EMC/OMCm and CBRs was −0,21. Both variables were randomly distributed, and symmetrically distributed. It was assumed that the function was normally distributed since the distribution of EMC/OMCm was normal, and for the distribution of CBRs uniform and normal distributions gave similar results. As a check on this assumption, the distribution was assumed as Beta, and the result was similar to that for the assumption of normality (discussed in more detail in Emery, 1985a).

Example Find the statistical properties of modulus of G7E materials

The predicted EMC/OMCm is 1,0. The probability density function of EMC/OMCm is defined by a normal distribution with a mean for this example of 1,0, and a standard deviation of 0,261. The probability density function of CBRs of G7E materials is defined by a uniform distribution with a mean of 11 and a standard deviation of 2,31. The two are correlated with r = −0,21.

The relationship between modulus and mc/OMCm (which in this case will be EMC/OMCm) and CBRs was found earlier:

E = 187 e[−0,59(mc/OMCm)] CBRs{0,20} (from equation 37)

The statistical properties of both mc/OMCm and CBRs were given above. Now combining these two probability density functions into the form of equation (37) would give a complex and almost unmanageable probability density function for modulus. It would be geometrically defined by a complex three dimensional shape; Figure 19 shows the elements of this but no attempt was made to draw it. Instead some of the statistical properties of modulus were found using the point estimate method. Essentially, when using the point estimate method for a function of two variables, the function is evaluated at four points, each representing some combination of each mean plus or minus one standard deviation.

It was slightly more complicated here because the two variables were correlated, but the essence is unchanged. Instead of the points being plus or minus one standard deviation, they are plus or minus a factored standard deviation, on the basis of:

P11 = P22 = (1 + (-0,21))/4 = 0,1975
P12 = P21 = (1 - (-0,21))/4 = 0,3025

modulus, the two variables in the function were mc/OMCm (or EMC/OMCm) and CBRs. The first point (E++) was equation (37) evaluated for EMC/OMCm = 1,2109 (mc/OMCm+) and CBRs = 13,8588 (CBRs+); the second point was equation (37) evaluated for EMC/OMCm = 1,2109 (mc/OMCm+) and CBRs = 9,1335 (CBRs-); and so on. Thus the four points were E++, E+-, E-+, E--.

First the upper and lower values of EMC/OMCm and CBRs were found:

EMC/OMCm+ = 1 + 0,261(0,1975/0,3025)[0,5] = 1,2109
EMC/OMCm- = 1 - 0,261(0,3025/0,1975)[0,5] = 0,6770
CBR+ = 11 + 2,31(0,3025/0,1975)[0,5] = 13,8588
CBR- = 11 - 2,31(0,1975/0,3025)[0,5] = 9,1335

Then equation (37) was evaluated at the four combinations of these points:

E++ = 155 MPa
E+- = 142 MPa
E-+ = 212 MPa
E-- = 195 MPa

From the values of the function at these four points, the mean and
deviation of modulus of G7E materials were found:

E = 0,1975(155) + 0,3025(142) + 0,3025(212) + 0,1975(195)
 = 176 MPa

Variance of E = 0,1975(155)[2] + 0,3025(142)[2] + 0,3025(212)[2]
 + 0,1975(195)[2] - 176[2] = 974

Standard deviation of E = 31,2

Table 25 Probability density function of modulus of G7E materials

| EMC/OMCm | Modulus (MPa) of G7E materials Mean | Standard deviation |
|---|---|---|
| 0,50 | 237 | 39,2 |
| 0,75 | 204 | 36,6 |
| 1,00 | 176 | 31,2 |
| 1,25 | 152 | 26,9 |
| 1,50 | 131 | 23,5 |
| 1,75 | 113 | 20,2 |
| 2,00 | 98 | 16,1 |

The probability density function of G7E modulus was used to find the probability that the subgrade modulus would be less than that needed for the design traffic if a G7E material was used in place of a G7 material in the subgrade. The minimum subgrade modulus for the design traffic was found for each of the five designs using Figure 18 (Table 26). Within each traffic class, there was a wide range between minimum and maximum design traffic, and values were found for both.

Table 26 Minimum subgrade modulus for design traffic

| Design | Minimum design traffic ME80s | Subgrade modulus MPa | Maximum design traffic ME80s | Subgrade modulus MPa |
|---|---|---|---|---|
| A E4 | 12 | <40 | 50 | 66 |
| B E3 | 3 | <40 | 12 | 94 |
| B E2 | 0,8 | 110 | 3 | 130 |
| C E1 | 0,2 | 78 | 0,8 | 102 |
| C E0 | 0,05 | 78 | 0,2 | 96 |

The probability that the actual subgrade modulus would be less than the required subgrade modulus was simply the area under a normal distribution such that:

$$z = (\text{actual} - \text{mean})/\text{standard deviation} \quad (38)$$

Example Find the probability that the G7E modulus is insufficient to carry the design traffic

For the C E1 design, and EMC/OMCm = 1,0, the probability that the G7E modulus is insufficient to carry the maximum design traffic of 0,8 ME80s is the probability that it is less than 102:

$z = (102 - 176)/31,2 = -2,37$
For $z = -2,37$, probability = 0,0089.

For all five designs the probability of the G7E modulus being insufficient for the minimum and for the maximum design traffic was found (Figure 20). As the predicted EMC/OMCm ratio increased to a high level, the probability increased substantially. Predicted EMC/OMCm was partially a function of climate and increased with increasingly wet climates. In the wetter climates the probability of failure associated with using unsoaked CBR design and G7E material in place of G7 material, would be greater than in the drier climates, which was expected from engineering experience. Since predicted EMC/OMCm was also partially a function of material type, the probability of failure increased with increasingly plastic materials.

It can be seen that some pavement structures were more sensitive than others to subgrade moisture. The heavy A E4 structure appeared to be almost completely insensitive to subgrade moisture. The B E2 structure with a granular basecourse and a thin cemented subbase was very sensitive to subgrade moisture; the use of unsoaked strength criteria would appear to be limited for this type of structure. The lighter class C structures were not seriously affected by subgrade moisture until it increased beyond an EMC/OMCm ratio of 1,5.

## 6.5 Economic Comparison Of Unsoaked Subgrade Design With Soaked Subgrade Design

The costs of pavements built using the proposed unsoaked subgrade CBRomc criteria were compared with the costs of building using the existing soaked subgrade CBRs criteria. The costs compared were total costs over the analysis period, discounted to present day values. These were a function of the cost of construction, time of terminal condition, the type of premature rehabilitation, and the cost of rehabilitation.

Cost of construction
The cost of construction varied with subgrade preparation. For subgrades with a soaked CBR greater than 15, there was no cost difference, and no calculations were done. For subgrades with a soaked CBR between 7 and 15, and CBRomc greater than 15, the subgrade construction cost for unsoaked design was limited to ripping and recompacting the insitu material; for soaked CBR design it was importing 150mm of G7, and ripping and recompacting insitu material. The construction cost of the upper pavement layers was the same for both cases. The case where the subgrade soaked CBR was less than seven was not calculated here in order to simplify the analysis.

Time of terminal condition
The terminal condition defined in TRH 4 (NITRR, 1985b), with depth of ruts, crack types, and length of road exceeding

limits, has been used here. Three discrete times to terminal condition were used in order to limit the analysis to a reasonable level: terminal condition reached before minimum design traffic was carried, terminal condition reached between minimum and maximum design traffic, and terminal condition at maximum design traffic.

The cumulative equivalent traffic was modelled according to TRH 4 (NITRR, 1985b). It was assumed that each design would carry cumulative equivalent traffic equal to its maximum design traffic by the end of the structural design period. The E80 growth rates for road categories A, B, and C were taken as ten, eight and six percent respectively, and the structural design periods as 20, 15, and 10 years respectively (both as recommended by TRH 4). It was assumed that the roads were two lanes wide with no shoulders to simplify the analysis, although the provision of sealed shoulders not material to the analysis; accordingly predicted equilibrium moisture content was not factored for the unsealed shoulders. The cost/benefits of sealed shoulders are discussed in a later section. For terminal condition before minimum traffic, the time of terminal condition was taken as half the time to minimum traffic. For terminal condition between minimum and maximum traffic, the time of terminal condition was taken as half the structural design period. Given the uncertainties associated with prediction of traffic growth, the simplicity of these assumptions was acceptable. A sensitivity analysis showed the result to be reasonably insensitive to changes in assumed time of terminal condition.

Example Find the time and traffic at terminal condition For design C E1, the maximum cumulative equivalent traffic is 0,8 ME80s. The structural design period is 10 years. For a growth rate of 6 percent, the traffic growth is as follows:

| Year | E80s per day | Cumulative ME80s |
|---|---|---|
| 1 | 160 | 0,06 |
| 2 | 169 | 0,12 |
| 3 | 179 | 0,19 |
| 4 | 190 | 0,26 |
| 5 | 202 | 0,33 |
| 6 | 214 | 0,41 |
| 7 | 227 | 0,49 |
| 8 | 240 | 0,58 |
| 9 | 256 | 0,68 |
| 10 | 272 | 0,80 |

Cumulative traffic equal to half minimum design traffic (0,1 ME80s) should occur between year one and two (taken as 1,7 years), and at half the structural design period (5 years), the cumulative traffic was taken as 0,33 ME80s.

Type of rehabilitation
The type of rehabilitation required at the premature terminal condition was found by considering the mode of terminal condition for each of the five designs, based on their mechanistic analysis. Terminal condition of the lighter designs (C E0, C E1, B E2) was generally characterised by rutting due to excessive subgrade vertical microstrains. For the heavier pavements, there was cracking of the bound layers due to excessive tensile strain, and these cracks had spread to the surface.

The rehabilitation options were standardised for the different designs in order to hold as many variables constant as possible; they were limited to either a bituminous overlay or a seal. The exclusion of reconstruction as a rehabilitation option was reasonable because of the high delay costs and thus road user costs associated with major reconstruction (TRH 12, NITRR, 1984). It can give a lower total cost to the community if the road is overlaid rather than reconstructed, even if actual cost of the overlay is higher. Standardisation on overlay/seal meant that differences in road user costs between options would be minimised and so need not be considered. The effect of this standardisation on total costs was small because they were not very sensitive to small changes in rehabilitation costs.

For terminal conditions due to rutting, the rehabilitation was aimed at reducing deflection. It was assumed that premature terminal condition occurred because the subgrade had wetted up (i.e. the field moisture content was now greater than the predicted equilibrium moisture content for whatever reason) and its modulus had decreased. The deflection associated with premature terminal condition was estimated by re-running the mechanistic analysis with varying subgrade moduli until the predicted equivalent traffic corresponded to the cumulative traffic at premature terminal condition. The surface deflection between the wheels for that subgrade modulus was taken to be the maximum annual deflection such as used in Figure 11, TRH 12 (NITRR, 1984).

No allowance was made for seasonal variation of deflection (if indeed it exists - Smith and Hewitt, 1984), nor for the effect on upper layer moduli of a weak subgrade. To answer any case of seasonal variation of deflection, the pavement was assumed to be at its wettest since it had reached terminal condition prematurely, and deflection should be at its maximum. In the case of upper layer moduli, it was assumed that the pavement was constructed at optimum moisture content, and that the then unsoaked subgrade gave sufficient support for the upper layers to be constructed to their required densities and thus moduli. The subsequent weakening of the subgrade would degrade the upper layer moduli, but to simplify the analysis this was not allowed for.

Example Find the maximum annual deflection at premature terminal condition
For design C E1 and analysis times of half minimum design traffic and half structural design period, the cumulative traffic to terminal condition was 0,1 ME80s and 0,33 ME80s respectively. From repeated mechanistic analyses with subgrade moduli varying from 60 to 200 MPa, the surface deflections between the wheels were determined by means of computer simulation using linear elastic concepts:

| Subgrade modulus MPa | Cumulative traffic ME80 | Surface deflection mm |
|---|---|---|
| 60 | 0,04 | 0,80 |
| 80 | 0,22 | 0,65 |
| 100 | 0,75 | 0,55 |
| 120 | 1,3 | 0,48 |
| 160 | 3,6 | 0,39 |
| 200 | 6,0 | 0,34 |

The respective maximum annual deflections equivalent to the cumulative traffic at premature terminal condition were 0,73mm and 0,62mm.

The rehabilitation measures depend on the future traffic to be carried by the rehabilitated pavement. Taking this traffic as the outstanding balance between traffic carried already and the maximum design traffic, the design deflection of the rehabilitated pavement is found using the approach shown in Figure 11 of TRH 12 (NITRR, 1984). The deflection and traffic to date were used to establish the deflection-traffic load relation for that pavement (a line parallel to the Asphalt Institute design line). The appropriate rehabilitation overlay was then found from TRH 12, Table 1: recommended asphalt overlay remedial measures, and Figure 12: rehabilitation design chart.

Example Find the rehabilitation measure
For design C E1 with maximum design traffic of 0,8 ME80s, and premature terminal condition at 0,1 ME80s with a maximum annual deflection of 0,73mm.

The deflection-traffic load line for the pavement to date was found (Figure 21). The future traffic to be carried to reach the maximum design traffic was 0,8 ME80s - 0,1 ME80s = 0,7 ME80s. This gave a design deflection of 0,45mm.

For a measured deflection of 0,73mm and a design deflection of 0,45mm, the rehabilitation design chart showed the rehabilitation category to be D (Figure 22). The recommended overlay was 90mm nominal thickness (Table 1, TRH 12). The rehabilitated pavement design was checked using mechanistic analysis. This showed the 90mm overlay to be inadequate in terms of granular layer safety factors and/or subgrade vertical strain, and so a 115mm overlay was adopted which was satisfactory in those terms.

Cost of rehabilitation
The cost of rehabilitating the pavement is a function of actual rehabilitation cost, time of rehabilitation, real discount rate, road user costs, and salvage value. These have been discussed in detail in TRH 12 (NITRR, 1984) and amplified in Jordaan (1985) and their principles will not be repeated here.

All rehabilitation costs were estimated from present day unit rates (Figure 23), and discounted to a present worth of cost. The unit rates used were an amalgam of unit rates from an example in draft TRH 4 (NITRR, 1980) updated to 1985, and 1985 CPA unit rates; however no particular significance was attached to the source of unit rates. These vary considerably from Province to Province, within a Province, with time etc., and repeated analyses using different unit rates was not warranted here. The discount rate was fixed at eight per cent, which is between the six per cent recommended by the South African Treasury (Jordaan, 1985) and the ten per cent suggested in TRH 4 (NITRR, 1985b). No allowance was made for inflation, as the simplifying assumption was made that all cost items escalate at the same rate. The analysis period was selected for the convenience of calculating salvage value (which was done here by bringing alternatives to the same end condition); it varied between 15 and 20 years. Road user costs were taken as equal for all options, and were not included. Maintenance costs were included to the extent of resealing cracked surfacings and periodic resurfacing of bituminous surfaces. The surfacing lives were estimated from TRH 4, and generally ranged from 9 to 12 years.

Total cost over the analysis period

The total cost over the analysis period was found using a Bayesian probability approach. For design with the proposed unsoaked strength criteria, a decision tree was constructed using the probabilities found above (Figure 24). A second decision tree was constructed assuming existing soaked strength criteria were applied. Since the risk varied with predicted moisture, the costs of each outcome were summed and a total present worth of cost found for a range of ratios of equilibrium moisture content to optimum moisture content (EMC/OMCm) ratios (Table 27). The full calculations for each design have not been given here for reasons of space, but are available in Emery (1985c), and an example is given below.

Example Find the total costs over the analysis period
For design C E1, the probabilities of premature terminal

condition were found at the two analysis times: equivalent traffic of 0,1 ME80s and 0,33 ME80s. At each of these times, there were two options: distress (probabilities P1 or P3) and no distress (probabilities P2 or P4), such that:

P1 + P2 = P3 + P4 = 1 (39)

The probabilities vary with the ratio of predicted moisture content to optimum moisture content, and so are given for a range of these:

| EMC/OMCm | Cumulative equivalent traffic 0,1 ME80s | | 0,33 ME80s | |
|---|---|---|---|---|
| | P1 | P2 | P3 | P4 |
| 0,50 | 0,0000 | 1,0000 | 0,0003 | 0,9997 |
| 0,75 | 0,0003 | 0,9997 | 0,0023 | 0,9977 |
| 1,00 | 0,0008 | 0,9992 | 0,0081 | 0,9919 |
| 1,25 | 0,0030 | 0,9970 | 0,0285 | 0,9715 |
| 1,50 | 0,0119 | 0,9881 | 0,0986 | 0,9014 |
| 1,75 | 0,0418 | 0,9582 | 0,2638 | 0,7362 |
| 2,00 | 0,1075 | 0,8925 | 0,5504 | 0,4496 |

The present worth of cost of each measure (expressed in terms of rand per square metre) was as follows:

Construction costs for S2/100G4/150G5 and subgrade:
proposed unsoaked criteria = 2,10 + 2,00 + 2,25 + 0,30 = R6,65/$m^2$
existing soaked criteria = 2,10 + 2,00 + 2,25 + 1,20 + 0,30 = R7,85/$m^2$

Rehabilitation and maintenance costs:
115mm overlay at 1,7 years = 12,08 $(1 + 0,08)^{-1.7}$ = R10,60/$m^2$
60mm overlay at 5 years = 6,30 $(1 + 0,08)^{-5}$ = R4,29/$m^2$
10 years = 6,30 $(1 + 0,08)^{-10}$ = R2,92/$m^2$
20 years = 6,30 $(1 + 0,08)^{-20}$ = R1,35/$m^2$
Single seal at 10 years = 1,40 $(1 + 0,08)^{-10}$ = R0,65/$m^2$
20 years = 1,40 $(1 + 0,08)^{-20}$ = R0,30/$m^2$

The present worth of the options for the C E1 design have been shown in the previous Figure. The total cost is the sum of the probabilities multiplied by costs. For an EMC/OMCm ratio of 1,50, the probabilities were also shown on

Figure 24, and so the present worth of the unsoaked (i.e. subgrade chosen by CBRomc) design was:

0,0119x0,5x0,5x21,52 + 0,0119x0,5x0,5x20,47 +
0,0119x0,5x0,9x19,25 + 0,0119x0,5x0,1x18,20 +
0,9881x0,0986x0,3x0,4x15,21 + 0,9881x0,0986x0,3x0,6x14,16 +
0,9881x0,0986x0,7x0,9x12,94 + 0,9981x0,0986x0,7x0,1x11,89 +
0,9881x0,9014x0,6x0,6x10,92 + 0,9981x0,9014x0,6x0,4x9,87 +
0,9881x0,9014x0,4x0,9x8,65 + 0,9881x0,9014x0,4x0,1x7,60
= R10,20/m[2]

The present worth of the soaked CBR design was:
0,6x0,6x12,12 + 0,6x0,4x11,07 + 0,4x0,9x9,85 + 0,4x0,1x8,80
= R10,92/m[2]

Table 27 summarises the costs of each of the five unsoaked designs over a range of predicted moisture contents, and compares these with costs of soaked designs.

Table 27 Cost of unsoaked designs for a range of predicted moisture contents

| Predicted mc/OMCm | Total present worth of cost (R/m[2]) | | | | |
|---|---|---|---|---|---|
| | C E0 | C E1 | B E2 | B E3 | A E4 |
| UNSOAKED | | | | | |
| 0,50 | 8,65 | 9,72 | 11,19 | 14,69 | 28,23 |
| 0,75 | 8,65 | 9,73 | 11,26 | 14,69 | 28,23 |
| 1,00 | 8,67 | 9,76 | 11,44 | 14,70 | 28,23 |
| 1,25 | 8,71 | 9,85 | 12,00 | 14,72 | 28,23 |
| 1,50 | 8,90 | 10,20 | 13,39 | 14,80 | 28,23 |
| 1,75 | 9,43 | 11,07 | 15,63 | 15,02 | 28,24 |
| 2,00 | 10,52 | 12,62 | 18,04 | 15,44 | 28,26 |
| SOAKED | | | | | |
| - | 9,85 | 10,92 | 12,38 | 15,89 | 29,43 |

The cost of adopting the proposed unsoaked strength criteria in the subgrade was compared with the cost of the soaked strength criteria for each design (Figure 25). The total cost of the proposed unsoaked subgrade strength criteria was less for all pavements when the ratio of predicted moisture content to optimum moisture content was less than 1,25. The

B E2 design with the granular basecourse over the thin cemented subbase was sensitive to subgrade support as expected, and if the predicted EMC/OMCm ratio was above 1,25, it was less expensive to use soaked design. For lighter designs with granular base and subbase, the unsoaked subgrade strength criteria would cost less except where the predicted EMC/OMCm ratio was very high. For the heavier pavements (A E4, B E3), the proposed unsoaked subgrade strength criteria was less expensive for all realistic moisture conditions. This was understandable because the base and subbase contribute much more to the heavier designs than does the subgrade.

Effect of sealed shoulders

The effects of the zone of seasonal variation of moisture content at the edge of the pavement were not included in the above cost comparison, other than by factoring the predicted EMC/OMCm ratio if the shoulders were unsealed. The cost/benefits of sealed shoulders are considered here briefly in terms of the effects of shoulder sealing on the adoption of soaked or unsoaked design. Only the full-width construction case is considered to simplify the analysis, and the analysis is neither a rigorous one nor is it intended to be a warrant for sealing shoulders.

The effect of sealed shoulders would to be to reduce the mc/OMCm ratio of the layers relative to roads with unsealed shoulders (provided the sealed shoulders were of sufficient width as discussed earlier). If this meant that unsoaked design could be used in place of soaked design, the effect would be to save about R9000 per kilometre of two lane road (at the unit rates used here) for all five designs used here. The extra cost of sealed shoulders by comparison, assuming the shoulder has been constructed unsealed and only needs sealing, is about R4000 per kilometre for both shoulders. If box construction had been used and there were no unsealed shoulders constructed, the cost of sealed shoulders would be much higher, but then the cost/benefit of box versus full-width construction would affect the analysis and this lies outside the scope of this bulletin.

## 6.6 Summary of the application of the proposed unsoaked strength criteria

The proposed unsoaked strength criteria are restricted in application to subgrades of the same types of pavement which were sampled in the various moisture studies: rural road drainage, deep water table, bitumen surfacing, and South African standards of construction, quality control, drainage, and maintenance. Furthermore the application of these criteria to pavements with a granular basecourse and a thin cemented subbase is not recommended at this stage because of those pavements sensitivity to subgrade strength. The moisture predicted by equation is moisture in the equilibrium zone of the pavement. For roads with unsealed shoulders, this predicted subgrade moisture should be increased by a factor of 1,25 in the lane(s) adjacent to a shoulder. For roads with sealed shoulders of a minimum width of 1,0 metres for traffic classes E1 and E2, and 1,2 metres for traffic classes E3 and E4, the predicted moisture does not need to be increased. An appropriate allowance should be made if the shoulder is unsealed when predicting the moisture in calculating probability of failure.

The application can be summarised as two steps:

1) Predict the ratio of equilibrium to optimum moisture content on the basis of laboratory results (equations 15 to 18) and Thornthwaite's moisture index (Figures 11 and 12). If the pavement is to have sealed shoulders (at least 1m wide for design traffic less than 3 ME80s, and 1,2m wide for design traffic above that), factor the predicted EMC/OMCm ratio by 1,0. If the shoulders are unsealed, factor the predicted EMC/OMCm ratio by 1,25 (if the road has more than one lane, this only applies to the lane(s) adjacent to a shoulder). If the factored predicted EMC/OMCm is less than 1,7 (except for granular base/thin cemented subbase designs), the unsoaked subgrade strength criteria are recommended.

If the subgrade of a proposed pavement has been split into areas on the basis of centre line CBR-values, the design EMC/OMCm for the areas should be taken as the upper 10th percentile, in the same way and for the same reason as design CBR is taken as the lower 10th percentile.

2) Select subgrade preparation according to Table 28.

Table 28 Preparation of subgrade (proposed method)

| | | | | |
|---|---|---|---|---|
| Soaked CBR | 3 - 7 | 7 - 15 | 7 - 15 | >15 |
| CBRomc | - | < 15 | > 15 | n.a. |
| Add layers | 150mm G7E | 150mm G7E<br>150mm G9 | nil | - |
| Insitu treatment | | rip and recompact | | |

where G7E is a G7-equivalent material meeting G7 Atterberg Limit criteria but with CBRomc greater than 15.

Typical cost savings
For a two lane road with 3,7m lane widths and 1,3m sealed shoulders and a low quality insitu subgrade (7<CBRs<15, CBRomc>15), the immediate construction savings of the proposed unsoaked criteria are about R12 000 per kilometre (at the costs assumed here). The present

worth of the total savings over the analysis period is about R10 000 per kilometre. With estimated costs assumed here (excluding drainage etc.) ranging from R98 500 per kilometre to R294 430 per kilometre for the five designs analysed, the percentage savings range from 3,4 to 10 per cent by using the proposed unsoaked strength criteria.

Example Selection of soaked or unsoaked strength criteria for the subgrade of a rural road pavement

The existing road from Hanover to Middelburg has to be rebuilt with a totally new alignment. This will be a two-lane rural road with unsealed shoulders. Information is available on the current traffic and the centre line subgrade CBR-values.

Road category and traffic The procedures for choosing road category and traffic class are as laid out in TRH 4 (NITRR, 1985b). This was taken as a Category C road; the analysis period and structural design period were 20 years and 10 years respectively; the design traffic class was E2.

Materials By visual inspection of the CBR values, three subgrade areas were delineated:

Subgrade area 1: CBRs = 12;10;7;8;11;14
CBRomc = 24;19;16;18;21;26

Subgrade area 2: CBRs = 20;23;20;18;19;15;16
CBRomc = 35;39;34;31;32;27;29

Subgrade area 3: CBRs = 10;12;10;10;8;9;8;7
CBRomc = 19;26;22;20;18;19;18;16

The lower 10th percentile CBR was found as shown in TRH 4, except that CBRomc tests were done to complement the soaked CBR tests:

| Subgrade area | CBR (%) mean | s.d. | Lower 10% | Design CBR |
|---|---|---|---|---|
| 1 soaked | 10,3 | 2,6 | 7,0 | 7 - 15 |
| omc | 20,7 | 3,8 | 15,9 | > 15 |
| 2 soaked | 18,7 | 2,7 | 15,3 | > 15 |
| omc | 32,4 | 4,0 | 27,3 | > 15 |
| 3 soaked | 9,3 | 1,6 | 7,2 | 7 - 15 |
| omc | 19,8 | 3,1 | 15,8 | > 15 |

In subgrade areas one and three, the use of unsoaked design may be possible depending on the predicted EMC/OMCm ratios of the materials. In subgrade area two, the use of unsoaked design has no benefit since the insitu material is already of a high quality.

Moisture For each of the CBR samples in the two subgrade areas where unsoaked design may be possible, the ratio of equilibrium to optimum moisture content was predicted. Thornthwaite's moisture index was taken as -30 from the Thornthwaite Im map.

Subgrade area 1: EMC/OMCm = 0,82;0,83;0,98;0,92;0,78;0,63
Subgrade area 2: not required
Subgrade area 3: EMC/OMCm =
0,90;0,91;0,93;0,90;0,95;0,98;1,04;0,95

The upper 10th percentile EMC/OMCm ratio was found for each subgrade area and the appropriate shoulder factor included:

| Subgrade area | EMC/OMCm mean | s.d. | Upper 10% | Shoulder factor | Design EMC/OMCm |
|---|---|---|---|---|---|
| 1 | 0,83 | 0,12 | 0,98 | 1,25 | 1,23 |
| 2 | not required | | | | |
| 3 | 0,95 | 0,05 | 1,01 | 1,25 | 1,26 |

Since the factored predicted EMC/OMCm was substantially less than 1,7 in the two areas where unsoaked design was being considered, the use of unsoaked design principles with the CBRomc test for the subgrade was accepted without further analysis for all possible pavement structures except those with granular base/thin cemented subbase.

## Application to other pavement design methods

There is considerable scope for the application of unsoaked design to other design methods in all pavement layers using the methodologies developed above. The application of unsoaked design to the TRH 4 method (NITRR, 1985) is probably limited to the subgrade because TRH 4 already incorporates unsoaked design in the basecourse and subbase through its use of effective moduli of upper layer unbound materials in an unsaturated state (Maree and Sutton, 1984), which were validated by field testing using the HVS (Freeme, 1983). This explains the small potential savings of only 3,4 to 10 per cent for unsoaked design in the TRH 4 method. However the potential savings for other design methods used in southern Africa are greater. Savings in stabilization and/or crushed stone base are possible with unsoaked design due to a reduction in cover thickness. A wider range of materials may be used since a material with a soaked CBR of 30 might easily have a CBRomc of 45 or more, i.e. it changes from a selected subgrade or light subbase to a high grade subbase. Similarly a subbase material might become suitable for use in bases on the basis of CBRomc. This again could save stabilization costs or the costs of importing better material. Savings can be up to 25 per cent of the construction cost of typical rural roads, with the greatest savings being in the more expensive upper layers.

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## APPENDIX A Statistics

Development of the moisture prediction methods, the analysis of moisture, and the calculation of probability and risk relied heavily on a statistical approach. Many of the statistical methods used were routine and are discussed in standard textbooks (such as Roscoe, 1975 and Moroney, 1953). However certain of the procedures are not routine and these are outlined here.

Analysis of variance

Analysis of variance (ANOVA) was used for testing the hypothesis that two or more independent samples were drawn from populations having the same mean. ANOVA compares the mean square (equal to the sum of squares divided by its degrees of freedom) within the groups, and the mean square between the groups. These provide independent estimates of the same common population variance and can be tested by the F-ratio. In the twoway test, the data is organised in a double entry table and the mean square is found for between the rows, between the columns, and for the row-by-column interaction. In the oneway, two sample case, ANOVA is mathematically equivalent to the t-test.

Analysis of variance makes the assumption of homogeneity of variances (discussed in texts such as Scheffe, 1959, or Seeger 1966). The analysis is only robust with respect to this assumption if equal sample sizes are used. In this analysis, sample sizes were unequal which could have led to invalid results, so both Cochran and Bartlett-Box tests (Bennet and Franklin, 1954) were made to test for homogeneity of variances. Where it was not found, this is reported. Logarithmic or square root transformations of the data were occasionally made to force them to homogeneity of variance before doing an analysis of variance. In addition, the non-parametric Kruskal-Wallis test (Roscoe, 1975) was occasionally done on untransformed data. Another option was to perform the Welch test (Bennet and Franklin, 1954), which is a oneway ANOVA on estimated variances with reduced degrees of freedom, and this can tolerate non-homogeneity of variances.

Since moisture content was closely related to material type, it was important to correct for differences in material type before analysing the effects of factors such as surface drainage. This was particularly so where the effect of the factor being tested was numerically quite small, and moisture differences due to material variations could overshadow differences due to the factor being tested. To correct for material differences, covariance was used in the analysis of variance (many standard texts, for example Roscoe, 1975). By what was essentially a regression process, the covariates (often plasticity index, liquid limit, linear shrinkage, percentage passing the 0,075 mm sieve, and a climate term) were regressed with moisture content and the subsequent equation used to correct moisture content for material type. The resultant corrected value was almost a "dimensionless" wetness, better in this respect than even suction. This technique was powerful, elegant, and sensitive, but extraordinarily demanding of mathematical computation. Until the recent advent of powerful computers and statistical packages, it was beyond the use of researchers by sheer weight of computation.

Multiple regression analysis

Multiple regression analysis combines two or more independent variables into an equation for the prediction of a single dependent variable. An index of the relationship between the variables is given by the multiple correlation coefficient R or r (note that in SPSS the computed coefficient is always positive). The squared correlation coefficient is the proportion of the variance of the dependent variable accounted for by the predictor variables (many standard texts, for example Younger, 1979; Nie et al, 1975). Multiple regression was done here using the Statistical Package for Social Sciences (SPSS), and generally a stepwise approach was used to develop the equations (Nie et al., 1975; Hull and Nie, 1981). For all the equations, a statistical test comparing the lack-of-fit error (sum of residuals minus replicate sum of squares) to the replicate error, showed that there was no reason to doubt the adequacy of the equations at the 0,01 level of significance.

Probability distributions

Pearson discovered that the majority of continuous probability distributions, f(x), can be generated from a single differential equation by the proper selection of constants. In particular the types of distribution can be classified by the coefficients of skewness and kurtosis. These are the third and fourth moments about the mean respectively (the first moment is the mean itself, and the second moment is the variance). Skewness provides a measure of the degree of asymmetry of a distribution; the normal distribution is perfectly symmetrical and has a skewness of zero. Kurtosis measures the degree of peakedness of a distribution; the normal distribution has a kurtosis of zero in SPSS (other texts give the kurtosis of a normal distribution as three; the difference between them and SPSS is simply arithmetical).

The type of distribution (normal, Beta, gamma, log-normal etc) can be found from the kurtosis and square of skewness of the data. There are three main type of distribution. The type I curve is bounded at both ends (such as a Beta distribution), the type VI is bounded at one end (such as log-normal), and the type IV is unbounded in both directions (such as the normal distribution) (Harr, 1977).

The normal probability distribution is symmetrical, bell-shaped and asymptotic to the X-axis. It is a simple and useful distribution for many sets of data and is common to find in engineering. A number of material parameters used in geotechnical engineering appear to follow a normal distribution based on the chi-square test at the five per cent level of significance (Lumb, 1966; quoted in Harr, 1977). However other material parameters do not follow the normal distribution and moisture content is an obvious one of these. A normally distributed variable ranges from minus infinity to plus infinity, whereas moisture content can only range between zero and saturation. The distribution of moisture content is also skew with a single long tail towards high moisture contents. However the versatile Beta-probability distribution is well suited to describing moisture content and other geotechnical variables which do not fit the normal distribution.

The Beta distribution has the properties that it need not be symmetrical and it is limited in range. It can be described by the mean, standard deviation, minimum, and maximum. By evaluating the Beta function at a number of points between the minimum and maximum, the probability distribution may be found, and from the cumulative distribution the various percentage probabilities may be found. Several computer programmes were developed here to analyse the Beta distribution and cumulative probability (Emery, 1985a).

## Approximate methods

Approximate methods are used to estimate the mean and standard deviation of a function of one or more variables. Generally if
$Y = f(X1, X2, X3, . . Xn)$, and each of the variables has a known mean and standard deviation, the mean and standard deviation of the function Y can be found. Of the three available methods: Monte Carlo simulation, Taylor Series expansion, and Rosenblueth's point estimate method (McGuffey et al., 1981), the point estimate method was used here.

In this method, first proposed by Rosenblueth (1975), the probability density function of a variable X can be approximated by a two point probability mass function which consists of concentrations P+ and P- at X+ and X- respectively. If Y is a function of X, the statistics of the probability density function of Y are obtained by evaluating the function Y(X) at X+ and X-. The accuracy is as good as that of the Taylor series approximation, and it is quicker to use than Monte Carlo simulation. In its simplest form, the mean and variance of the probability density function of Y(X) are:

mean E(Y) = P+Y+ + P-Y-
variance Var(Y) = P+Y+[2] + P-Y-[2] - E(Y)[2]

If X is symmetrically distributed, P+ = P- = 0,5, and X+ is one standard deviation above the mean, X- is one standard deviation below the mean. The input variables do not have to symmetrical, nor do they have to be uncorrelated, nor does the distribution have to be normal; there are extensions to the method to handle these situations (Grivas, 1984), although the method becomes slightly more involved.

If Y is a function of two correlated, symmetrically distributed, random variables (correlation coefficient = r) such that
Y = f(X1, X2), it is evaluated for Y11, Y12, Y21, Y22 where:

Y11 is the function at X11, X21
Y12 is the function at X11, X22
Y21 is the function at X12, X21
Y22 is the function at X12, X22

This was the case in this bulletin. Y is evaluated at:

X11 = mean{X1} - std.dev{X1} (p12/p11)[0,5]
X12 = mean{X1} + std.dev{X1} (p11/p12)[0,5]
X21 = mean{X2} - std.dev{X2} (p22/p21)[0,5]
X22 = mean{X2} + std.dev{X2} (p21/p22)[0,5]

where:
p11 = p22 = (1 + r)/4
p12 = p21 = (1 - r)/4

The mean of the function Y (mean{Y}) is the summation of Y{ij}p{ij}, and the variance is the summation of (Y{ij} - mean{Y})[2] p{ij}. Here this was used to find the statistics of the probability density function of resilient modulus, which was a function of two variables: soaked CBR and moisture.

## APPENDIX B Details of sites

Details of the seasonal sites are given below. Details of the equilibrium moisture sites would be too lengthy to give here and can be found in the relevant references: Burrow (1975) for the Transvaal data, and Emery (1983a) for the other data. All the sites are shown in Figure B1.

| Seasonal site | Road no. | Location | Climate Thornthwaite Im | MAR (mm) |
|---|---|---|---|---|
| D2 | T67/0 | Groblershoop to Kimberley | −32 | 350 |
| CW1 | T30/0 | Villiersdorp to Worcester | −10 | 550 |
| G26 | T2/0 | George to Sinkasbrug | −10 | 700 |
| VD101 | T9/0 | Prince Albert Road to Matjiesfontein | −48 | 150 |
| N2 | M2 | Gingindlovu to Kwa Mbonambi | +40 | 1250 |
| N71 | M1 | Balgowan to Estcourt | +20 | 750 |
| D20 | T13/0 | Victoria West to Britstown | −40 | 230 |