rected structural number and Dynaflect deflection.

Empirical relations for predicting asphaltic concrete cracking were developed as a function of traffic, age, and one of the following: (a) Benkelman beam deflection; (b) Dynaflect deflection; or (c) corrected structural number. In the prediction of cracking it was shown that, if more than 10 percent of the area of the road is cracked, cracking will reflect through a slurry seal within one year. Further, the rate that reoccurring cracking develops following a slurry seal exceeds the progression rate associated with the original cracking. Therefore, the utility of using a slurry seal for resealing may be questioned.

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Performance, Design, and Maintenance Relationships for Unpaved Low-Volume Roads

A. T. VISSER AND W. R. HUDSON

Although paved roads are widely studied, unpaved roads are far more widely used throughout the world. Recently, problems have been encountered in transferring experience and technology with unpaved roads to environments other than those in which they were obtained. In addition, low available funding demands that these funds be used with maximum benefit, and this requires the use of pavement management system methodology. An approach for evaluating unpaved road performance and deterioration is developed. The method is based on an extensive study in Brazil, and equations for predicting roughness, rut depth, and gravel loss are developed. Important criteria for the passability of an unpaved road and a minimum gravel thickness to protect the roadbed are presented. The maintenance and design system (MDS) presented combines these relationships with user cost equations in a systematic manner, which permits an evaluation of the interaction of the factors. Most important, traffic was found to have the greatest influence on regraveling and blading strategies as well as on the total cost of unpaved roads. The MDS has been tested by comparing predicted and actual maintenance on the unpaved road network in the Bronkhorstspruit District of South Africa and excellent agreement was found, which signifies that on average the MDS developed for Brazilian conditions can be applied to South African conditions.

The important designed performance details of unpaved roads differ from those of paved roads. For example, in using granular materials for paved roads we try to remove almost all clay or plasticity from the material. On the other hand, surface gravels were better when they contained some plasticity. Thus, taking their cue from nature, road builders of ancient and modern worlds have added clay to sandy roads to make them stable and have added sand and gravel to clay surfaces to prevent them from rutting and becoming impassable in wet weather. The clay acts as a binder, whereas the sand and gravel particles bear on each other and resist traffic loads. Reference to this type of construction appeared as early as 1806 (1). Textbooks of the 1920s (2) contain descriptions of the design and construction of earth, sand-clay, and gravel roads, which reflects the experience that was then available in the use of these roadbuilding materials.

In recent years there has been a remarkable expansion of the world's paved-road networks. Nevertheless, unpaved roads, i.e., earth, sand-clay, and gravel-surfaced roads, constitute a major part of the network in most countries. In developed countries the proportion varies between 5 and 63 percent of the total, whereas in developing countries they range from 70 to 97 percent of the network (3). In South Africa, 75 percent of the rural road network was unpaved in 1980 (4).

The fact that maintenance and design procedures for unpaved roads are normally based on local experience poses a problem, since these procedures cannot be transferred directly to other environments and material types. In addition, investment and maintenance alternatives for paved roads are studied by using pavement management systems that were developed to achieve the best value possible for the available public funds (5). To apply pavement management concepts to gravel-surfaced roads, it is necessary to quantify the behavior and performance of unpaved roads. The problems that are addressed in this paper are twofold:

1. The need for rational experienced methods to

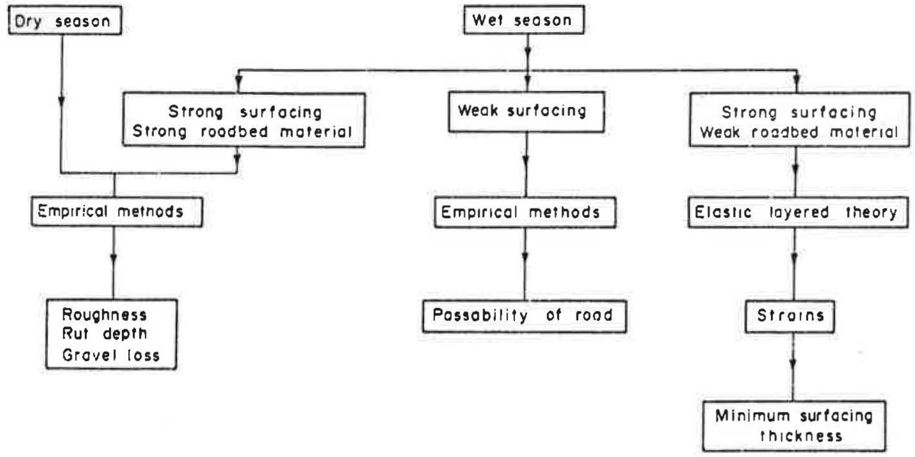


Figure 1. Methods of evaluating unpaved-road performance.

transfer technology to, and criteria from, one environment to another and

2. The need to evaluate unpaved-road administration or management to a more rational and technical level through pavement management system methodologies.

Note that a solution to the first problem is encompassed in the best solution to the second problem. In addressing these problems, the primary objective of the study was to develop a maintenance and design system (MDS) that would permit the evaluation of the relative economic benefits of alternative maintenance and design strategies.

This paper first describes the method of evaluating unpaved-road performance and then develops prediction equations for each component. The MDS contains these performance prediction equations as well as vehicle operating-cost and maintenance relationships. The results of the MDS are discussed in general terms, and the results of an application and test of the MDS in the Bronkhorstspruit District in the State of Transvaal, South Africa, are given.

EVALUATION OF ANALYSIS APPROACHES FOR PREDICTING UNPAVED-ROAD PERFORMANCE

There are four types of traffic-related deterioration that may be differentiated on unpaved roads. These are

1. Deterioration such as roughness, which occurs primarily as a surface phenomenon in the dry season;
2. Surface deterioration (corrugations and roughness) in the wet season even though good drainage exists and the surfacing and roadbed materials possess sufficient shear strength to withstand the imparted traffic load stresses;
3. Surface deterioration of a material that possesses low shear strength at moisture contents that are found during the wet season; and
4. Deformation of the roadbed during the wet season, which occurs where the roadbed material has a low shear strength [or California bearing ratio (CBR)] and the surfacing thickness is insufficient to reduce the deformations in the subgrade to within limits that the material can accommodate.

The interaction of these modes is illustrated in Figure 1, and a discussion of the possible mechanisms and a method of analysis follows.

Surface Deterioration in Dry Weather

The most prominent deterioration mechanisms in dry weather are

1. Wear and abrasion of the surface, which generate loose material and develop ruts;
2. Loss of the surfacing material by whip-off and dust;
3. Movement of loose material into corrugations under traffic action; and
4. Raveling of the surface in cases where there is insufficient binding power of the material to keep the surface intact, which often results in depressions that cause a rough ride.

At this time there are no theoretical models that can predict these deterioration mechanisms. Consequently, the most viable method of predicting performance is developing empirical models.

Surface Deterioration in Wet Weather

For a road where the shear strength of the surfacing and roadbed material is greater than the induced traffic stresses, the only deterioration that occurs is superficial. This type of deterioration is prevalent in regions where good drainage exists or good materials are found. The major mechanisms of deterioration under these conditions are

1. Environmental and traffic influences on surface erosion,
2. Wear and abrasion of the surface by traffic that causes rutting and loss of the surfacing material, and
3. Formation of potholes under traffic action.

The theoretical treatment of surface erosion has not yet reached a stage for general implementation (6). In addition, the formation of potholes is particularly dependent on the nature and frequency of depression and the frequency of wheel-load applications. The type of deterioration discussed in this section is most readily treated in an empirical manner analogous to that for the dry-season deterioration.

Deterioration of Weak Surfacing Layer in Wet Weather

A weak surfacing layer is one in which the shear strength under the existing operating conditions is less than the applied stresses, so shear failure occurs. It is possible that a vehicle will be able to pass through a stretch of road under these conditions, but it is very likely that the road will become impassable after a fairly small number of vehicle passages. Traditionally the CBR test was used to identify materials that resist shear failures. Plasticity analysis (7) is a theoretical

Table 1. Unpaved-road data summary.

Variable[a]	Mean	SD	Range Minimum	Range Maximum
Grade (%)	3.8	2.6	0.0	8.2
Curvature (1/rad) on curved sections	0.0039	0.0009	0.0025	0.0055
Road width (m)	9.8	1.09	7.0	12.0
Material properties				
Percent passing the 0.42-mm sieve	53	22	24	98
Percent passing the 0.074-mm sieve	36	24	10	97
PI (%)	11	6	0	33
Liquid limit (%)	32	9	20	62
ADT				
Passenger cars	88	64	11	288
Buses	7	7	0	29
Pickups	37	29	4	115
Two-axle trucks	56	93	1	435
Trucks and trailer combinations with more than two axles	15	18	0	66
Gravel loss				
No. of observations	604			
Time of observation relative to start of observation or regraveling (days)	238	211	0	1099
No. of bladings relative to start of observation or regraveling	2.3	3.3	0	23
Roughness measurements				
Roughness (QI) (counts/km)	117	61	15	445
No. of days since blading for last observation in each blading period	75	70	1	661
No. of vehicle passes since blading for last observation in each blading period	16 080	17 880	63	136 460
Rut-depth measurements				
Rut depth (mm)	11.1	8.6	0	75
No. of days since blading for last observation in each blading period	61	66	1	661
No. of vehicle passes since blading for last observation in each blading period	12 490	14 030	21	86 700

[a]Number of sections = 48.

analysis technique, but at this time it is only applicable to a single wheel pass and not to multiple passes, which normally cause impassability.

For a weak surfacing material--this usually applies to earth roads that comprise the existing in situ materials--it is virtually impossible to predict roughness development under these conditions of deformation. In fact, roughness has little meaning. The same is true for rut depth, and since the material is the in situ material, gravel loss is unimportant. The important factor is whether traffic would be able to use the road, and this can be predicted by empirical means.

Deterioration of Weak Roadbed Material in Wet Weather

Where a weak in situ soil exists, a pavement is placed to reduce the induced traffic stresses and strains. Frequently, however, the subgrade strain reduction provided by the pavement layers is insufficient, and overstressing of the subgrade occurs. Generally, deterioration under these conditions is a deformation of the roadbed material. This type of deterioration is predominant in areas of poor subsoil drainage. Investigations showed that the minimum gravel thickness cannot yet be determined from elastic-layer theory, and empirical methods had to be used.

DEVELOPMENT OF PREDICTION MODELS OF PERFORMANCE AND BEHAVIOR

Experimental Design of Empirical Study

A data base used to develop empirical relationships was collected in the Brazil study and reported in several working documents (8-13). The approach adopted for determining pavement deterioration relationships was to monitor existing in-service roads. A factorial experiment was designed that permitted the maximum use of data collected on a limited number of study test sections. The use of a factorial experiment also ensures that the full range of the independent variables, such as average daily traffic (ADT), grade, horizontal curvature, and surfacing-material type, found in practice is used in the analysis. The independent variables studied were selected from results of previous research on pavement performance, with major input from the Kenya study (6). Furthermore, the independent variables possessed controllable levels that covered the range found in the central plateau of Brazil where the study was conducted.

All of the study sections were basically 320 m long, but reduced section lengths were used when the desired section length with the specifically required characteristics could not be found. A total of 48 unpaved sections were studied, and a data summary is given in Table 1. Some of the terms will be clarified in the analysis sections.

Prediction Equation for Roughness

To relate pavement condition to vehicle operating costs, it is important to evaluate roughness. The roughness scale used in Brazil was based on data from a GM profilometer (9), which is a sophisticated and expensive system. The summary used, the quarter-car index (QI), relates analytically to the results of a Bureau of Public Roads roughometer, although it is not equal in magnitude. Maysmeters, which are simple, low-cost instruments mounted in a passenger vehicle or trailer, were used as routine roughness-measuring instruments. To ensure compatibility and stability, each Maysmeter was calibrated to the standard scale, which was termed the QI (9). The relationship between QI and the present serviceability index (PSI) measured with the modified PCA meter in South Africa is given below (14):

QI	PSI
10	4.28
30	3.03
50	2.14
70	1.52
90	1.07

The time period between bladings was called a cycle, since every time an unpaved road is bladed, the roughness is generally reduced, and the new deterioration during the cycle depends on the length of time since the previous blading. Consequently, deterioration was considered a function of the number of days since the last blading. In the Kenya study (6) the same approach was used, but in that study the researchers assumed that blading a road returned its roughness to some standard value. Inspection of the data collected in Brazil showed that the roughness measured after blading varied, so the assumption of a standard value was not appropriate.

The choice of the form of the deterioration model was based on considerations of the interaction of a vehicle with a road. As the first irregularities develop from traffic and weathering, increased dynamic forces are imposed on the road, with resultant deterioration and a rapid increase in roughness. At the upper end of the roughness scale, it is unlikely that a road will continue to increase in roughness but rather that at a very high roughness level the vehicles slow down and there is hardly any change in roughness. Very little data existed in the QI range of 300-450, and the exact modeling of the roughness development was therefore infeasible. Instead, an exponential curve that has an artificial upper limit at 450 QI was used.

The following independent variables were evaluated in the analysis of change of roughness with time:

1. Vertical grade of road;
2. Radius of curvature;
3. Liquid limit of surfacing;
4. Plasticity index (PI) of surfacing material;
5. Percentage of surfacing material passing the 0.42-mm sieve;
6. Percentage of surfacing material passing the 0.074-mm sieve;
7. ADT of each of the five vehicle classes: cars, pickups, buses, two-axle trucks, and other trucks;
8. Uphill or downhill lane;
9. Road width;
10. Wet or dry season;
11. Qualitative surfacing-type descriptors, e.g., laterite, quartzite, or clay; and
12. Time in days since the most recent blading.

The change in natural logarithmic value of roughness is given in the following exponential model:

$$\begin{aligned} LDQ = D\ [&0.4313 - 0.1705T2 + 0.001\ 159NC + 0.000\ 895NT \\ &-0.000\ 222\ 7NT \times G + S(-0.1442 - 0.0198G \\ &+ 0.006\ 21SV - 0.0142PI - 0.000\ 617NC)] \end{aligned} \tag{1}$$

where

- D = number of days in hundreds since last blading or since regraveling for gravel loss;
- T2 = surfacing-type dummy variable (T2 = 1 if surfacing is clay, T2 = 0 if laterite or quartzite);
- NC = average daily car and pickup traffic in both directions;
- NT = average daily bus and truck traffic in both directions;
- G = absolute value of grade (percent);
- S = season dummy variable (S = 0 if dry, S = 1 if wet);
- SV = percentage of surfacing material passing the 0.074-mm sieve; and
- PI = plasticity index of surfacing material (percent).

This model has an R^2-value of 0.26, and the sample size was 8276 observations. The standard error is 0.222 in terms of the logarithmic transformation. Thus, if Equation 1 predicts a change in roughness of 100, the true value of the change falls between 65 and 154 with 95 percent confidence. Care should be taken in the application of equivalency factors between the two vehicle types, since another analysis, in which the average daily traffic was substituted for the two traffic classifications, showed that the confidence interval was not meaningfully worse than in Equation 1, although statistically larger at the 95 percent level of confidence.

The model predicts an increase in roughness with time even in the absence of traffic. This follows normal experience. Traffic generates roughness, but on grades this is reduced for trucks, mainly because of lower speeds. In the wet season the development of roughness is lower than in the dry season, and on grades it is even lower, probably because of better drainage. During the wet season, surfacing-material characteristics are important, and experience indicates that a low percentage of fine material, which results in a low PI, is beneficial. To predict roughness after blading, the season during which blading occurred was studied in addition to the independent variables listed in the change of roughness analysis. The following model was developed:

$$\begin{aligned} LRA = 1.4035 &- 0.0239W - 0.0048SV + 0.016\ 94PI \\ &+ 0.6307LRB + 0.1499T1 + 0.3096T2 + 0.000\ 20NT \\ &+ 0.2056BS - 0.011\ 83PI \times BS \end{aligned} \tag{2}$$

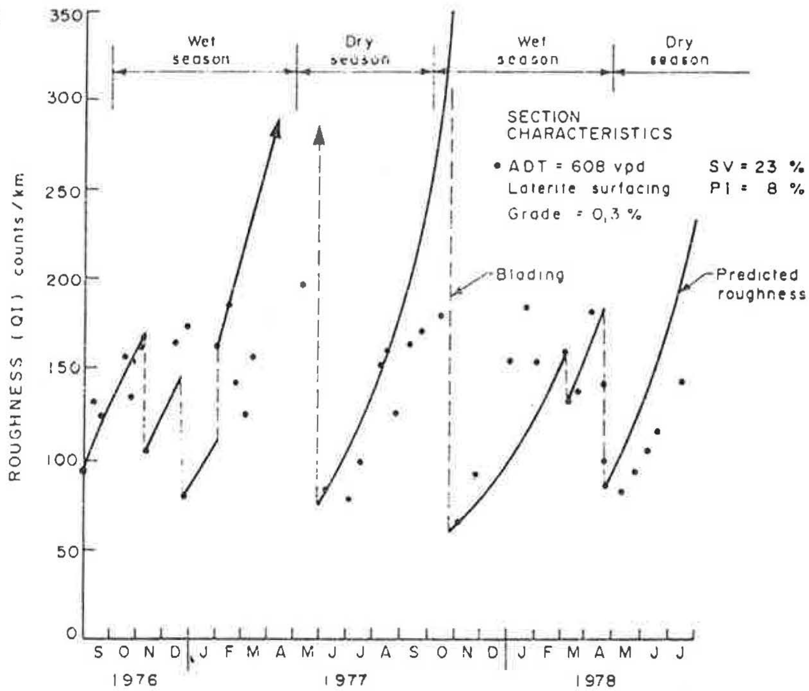
where

- LRA = natural logarithm of roughness (QI) (counts/km) after blading;
- W = road width (m);
- LRB = natural logarithm of roughness (QI) (counts/km) before blading;
- T1 = surfacing-type dummy variable (T1 = 1 if surfacing is quartzite, T1 = 0 if laterite or clay); and
- BS = dummy variable for season during which blading occurred (BS = 0 in dry season, BS = 1 in wet season).

A total of 1308 observations were used to develop the model, which has an R^2-value of 0.61. The standard error of the model, in logarithmic terms, is 0.34, which means that if the predicted roughness after blading is 100, the confidence interval is 52-194. This may seem high but is reasonable for such roads.

Roughness after blading is highly dependent on the roughness before blading, as would be expected. The higher the PI, the higher the roughness after blading because of the ready formation of a hard upper crust that is usually not disturbed during blading. Increasing the percentage of fine material reduces the roughness after blading because of the greater ease in spreading and cutting the surfacing material. The average daily truck traffic increases the roughness after blading, probably because of a higher degree of compaction of the upper part of the surfacing, which makes it more difficult to cut this material.

Figure 2. Measured and predicted roughness on section 251 under infrequent maintenance.



Roughness at any time during the season is determined as the exponential of the sum of the logarithm of the change in roughness over time and the logarithm of the roughness at time zero. The form of the model is shown in Figure 2, which represents the results on a specific section. Because of the absence of an international standard scale for roughness, it is not yet possible to compare the roughness models with those of other studies.

Prediction Equation for Rut Depth

Users of unpaved roads claim that deep ruts affect the safe operation of vehicles and could lead to accidents. In addition, prominent ruts act as drainage channels and prevent water from running off the roadway. The responsible agency therefore needs to know when to program maintenance to avoid such situations.

Rut depth was measured as the depression transverse to the axis of the road under a 1.22-m straight edge. Ruts were defined as those areas on the road demarcated by the absence of loose material, since field observations showed that most vehicles traveled in these areas. The rut-depth studies were aimed at predicting the rut depth at any time.

A deterioration cycle for rutting started immediately after a blading and continued until the next blading. Inspection of the data showed that, contrary to general belief, rut depth after blading was not zero, which agreed with the Kenya study (6) results. The rate of change in rut depth was hypothesized to be a function of a linear combination of the independent variables. This model form was selected since the development of rut depth appeared to be independent of the existing rut depth for the available data. In addition to the independent variables studied for roughness, the internal or external wheelpath was also considered. The dependent variable investigated was the change in rut depth in millimeters.

Three season descriptors were used, since an inspection of the data showed that at the start of the wet season a rapid decrease in the rut depth occurred. This continued for the first two months of the wet season, or until the section was bladed. This phenomenon occurred because drivers avoided the ruts where water was ponding, which caused the ruts to decrease. The significant variables for the change in rut depth with time are shown in the models for the three seasonal conditions:

Dry season (S1 = 0, S2 = 0):

$$DRD = D(9.78 - 1.033G - 0.192PI - 3.63T2 + 0.0302NC + 0.0198NT - 3.27RO - 3.04NT/R + 0.46G \times RO - 325.0L/R + 0.0364L \times SV) \quad (3a)$$

Transitional season (S1 = 1, S2 = 0):

$$DRD = D(-83.76 + 3.658G - 0.192PI - 3.63T2 - 0.1147NC + 0.1249NT - 3.27RO - 3.04NT/R + 0.46G \times RO - 325.0L/R + 0.0364L \times SV + 6.874W) \quad (3b)$$

Wet season (S1 = 0, S2 = 1):

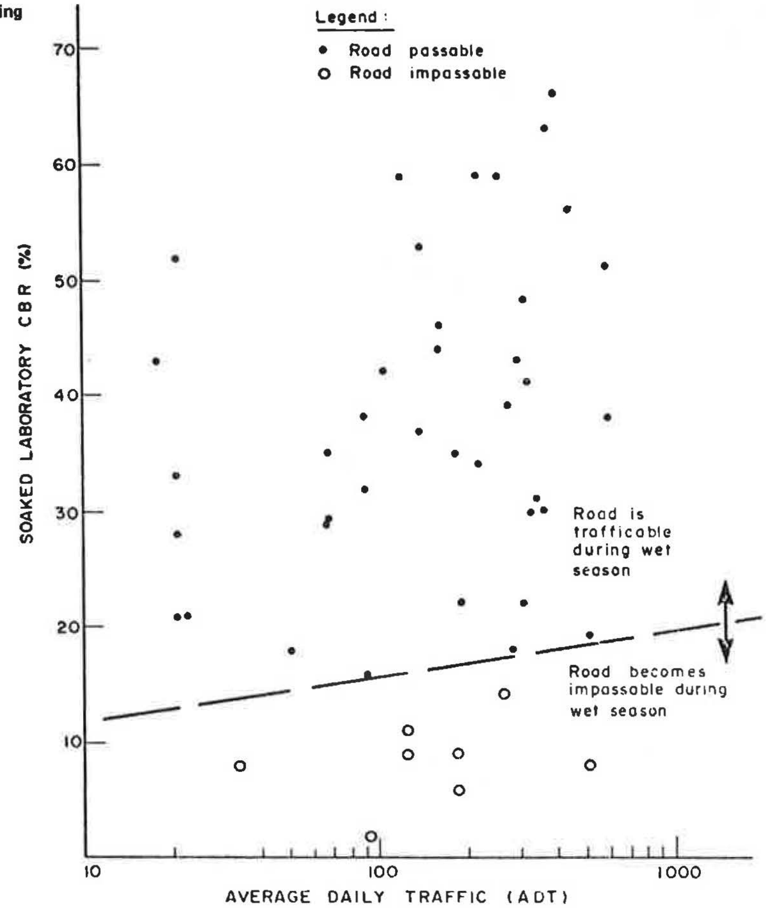
$$DRD = D(9.78 - 1.033G - 0.192PI - 3.63T2 - 0.0109NC + 0.0198NT - 3.27RO - 3.04NT/R + 0.46G \times RO - 325.0L/R + 0.0364L \times SV) \quad (3c)$$

where

- S1 = transition from dryto wet-season dummy variable: first two months of wet season or until first blading in this period (S1 = 1, otherwise S1 = 0);
- S2 = wet-season dummy variable: after first blading in first two months of wet season or after two months in wet season (S2 = 1, otherwise S2 = 0);
- DRD = change in rut depth with time (mm);
- RO = wheelpath dummy variable (RO = 0 for external wheelpath, RO = 1 for internal wheelpath);
- L = lane dummy variable (L = 0 for uphill lane, L = 1 for downhill lane); and
- R = radius of horizontal curvature (m).

This model has an R^2-value of 0.14 and a standard

Figure 5. Soaked laboratory CBR values for unpaved-road surfacing materials.



2. A maximum level dictated by the soaked CBR for stability and passability during the wet season and the limitation of dust problems.

Surfacing-Thickness Requirements

A pavement structure above the existing roadbed is generally provided to reduce the stresses and strains in the roadbed material to a level that the material can support. With a sufficient surfacing thickness, the pavement is strong enough that traffic deterioration is primarily surficial, and the road may be classified as "strong." The deterioration relationships previously developed then apply.

An attempt was made to develop thickness requirements from elastic-layer theory, but inconclusive results were found. Empirical methods had to be used, and a literature survey showed that a U.S. Army Corps of Engineers model (18) was the most up to date and gave realistic results when compared with observations on the study sections in Brazil.

The model is a function of the wheel load, tire pressure, number of load repetitions, terminal rut depth, and surfacing and roadbed material CBRs. Figure 6 shows the design curve for 10 000 load repetitions of a 40-kN wheel load, a tire pressure of 500 kPa, and a terminal rut depth of 75 mm. For any combination of surfacing and roadbed CBRs, the gravel surfacing thickness requirements can be determined. These thickness values also correspond to the experience in the Brazil project.

MAINTENANCE AND DESIGN SYSTEM FOR UNPAVED ROADS

The interaction and influences of the deterioration models developed earlier can be effectively evaluated only in a systems-analysis approach. The objective for the development of the maintenance and design model for unpaved roads was to evaluate the alternative regraveling and blading strategies within a system of constraints related to the road purpose and technical limitations for a specific road link. The basis of the evaluation considered was the total transport costs, including road-maintenance and road-user costs. Road construction costs were not considered since these costs will be the same for all alternative strategies. A simplified flowchart of the MDS is shown in Figure 7. In addition to the deterioration relationships developed above, the MDS also contains user-cost relationships, such as fuel consumption, interest and depreciation, tire consumption, and vehicle maintenance costs, developed in the Brazil study (19-21) but calibrated for South African conditions. To evaluate the behavior of the MDS, a sensitivity analysis was performed. Table 2 summarizes the influences of the various inputs on the results of the MDS in terms of the regraveling strategy and cost as well as the blading strategy and total cost. Those factors indicated to be either major or minor are influenced by certain circumstances.

The most important factors affecting the least-total-cost maintenance strategy and regraveling strategy were ADT and vehicle growth rate. Surfacing material properties influenced the blading strategy to a lesser degree and had no effect on the regraveling strategy. Other input factors, such as discount rate, analysis period, and road geometry, affect the total transport cost but not the blading or regraveling strategy, whereas ADT and vehicle growth affect the regraveling cost. The evaluation also showed that in general as little gravel as is

Figure 6. CBR design curves for Corps of Engineers model.

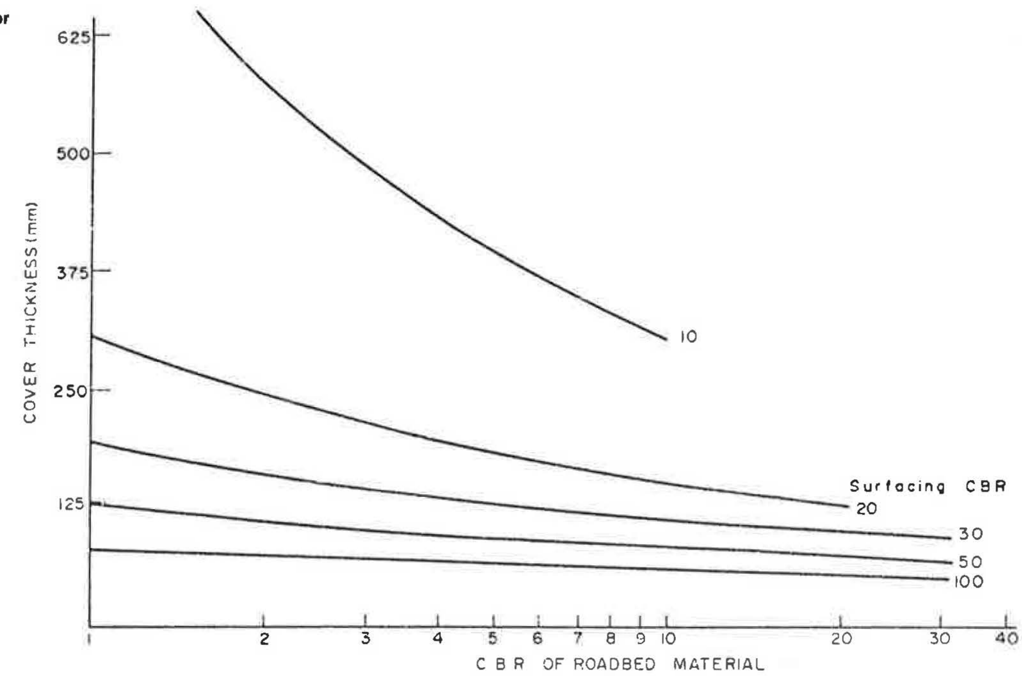
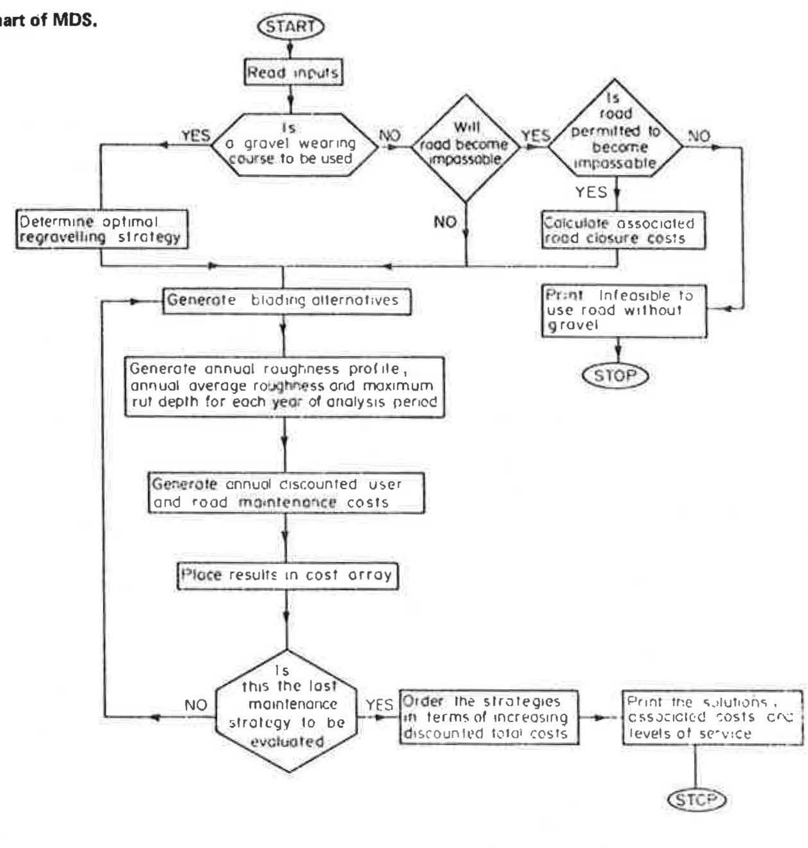


Figure 7. Simplified flowchart of MDS.



technically feasible should be placed when regraveling is necessary. Finally, in contrast to that of a paved road, the roughness of an unpaved road after construction or regraveling has no influence on the total transport cost. This is attributed to the effect of frequent bladings and a balance in the roughness that is achieved between traffic and maintenance.

Application of MDS to South African Conditions

The MDS was developed from information collected in other countries, and although these relationships were calibrated for South African conditions, it was essential to test the model under working conditions. The Bronkhorstspruit District in the Transvaal, which covers 677 km of gravel roads, was used

Table 2. Influence of various inputs on results of MDS.

Factor	Regraveling Strategy		Regraveling Cost		Blading Strategy		Total Cost	
	Major	Minor	Major	Minor	Major	Minor	Major	Minor
ADT	X		X		X		X	
Road geometry		X		X		X	X	
Gravel quality		X		X	X[a]	X[a]	X[a]	X[a]
Analysis period		X	X			X	X	
Cost of moving equipment		X		X		X		X
Discount rate		X	X[a]	X[a]		X	X[a]	X[a]
Vehicle growth rate	X		X				X	
Salvage value		X	X					
Roughness after construction						X		X

[a]Either major or minor.

for this investigation (22). The input information for the MDS was either extracted from files in the district or measured in the field, and the test consisted of comparing the predicted minimum total cost blading frequency and annual gravel loss with the blading frequency and volume of material used for regraveling during the 1980-1981 fiscal year.

The predicted maintenance and that which was executed compare favorably. For example, the predicted volume of regraveling material of 129 000 m^3 is in close agreement with the 138 500 m^3 actually applied. In terms of blading frequencies, certain links of the network were bladed more frequently than was economically desirable, whereas others were not bladed sufficiently. This finding is in agreement with the reports of the supervisory staff in the district. In a certain region, an extremely zealous operator produced more than expected, whereas in another region frequent equipment problems occurred, which resulted in insufficient blading of certain links.

As a further check, the roughness of the network was measured at the end of the 1981-1982 fiscal year, and it was assumed that the same maintenance level was applied as in the 1980-1981 period. Individual links could of course have any roughness value, dependent on the time since the previous blading. However, on the average, the predicted and measured roughness were expected to correspond. This was found to be true; the measured roughness was 63 QI, whereas the predicted value was 58 QI, which indicates excellent agreement. In addition, the measured roughness was compared with the maximum predicted. For this case the measured values were less than or equal to the predicted values within the 95 percent confidence band.

CONCLUSIONS

Unpaved-road deterioration relationships that were originally developed in Brazil have been shown to apply to at least average conditions in the South African area as found in the Bronkhorstspruit District (23). Furthermore, the conceptual evaluation of unpaved-road deterioration combined with cost relationships contained in the MDS have been shown to apply to South African conditions. In this way two major problem areas defined in this paper have been resolved. The necessary information is now available for transferring road-surface technology to other environments, and unpaved-road administration can be executed by using pavement-management methodology in the form of the MDS. This means that the economic implications of different policies can be evaluated in realistic ways.

RECOMMENDATIONS

In its current form, the MDS can be effectively used for the planning of the maintenance and the regraveling of unpaved roads. However, the present format requires the input of information for each roadway link, and the program must be modified so that the information can be read from a data base. For budgeting purposes, as well as for determining the effects of alternative fund allocations, a network optimization subroutine is needed in the MDS and will be added soon. The output from this subroutine can also assist in determining the budget levels that satisfy marginal benefit/cost ratios normally used by the public sector.

In order to apply these results under even wider applications, it is necessary to verify the validity of the deterioration prediction equations on individual road sections. This work should be undertaken in several countries of the world simultaneously.

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Application of an Impact Test to Field Evaluation of Marginal Base Course Materials

BADEN CLEGG

A procedure for selection of base course materials that involves the use of a rapid in situ test based on an instrumented modified AASHO compaction hammer is discussed. Results of this test are compared with those of more common tests such as California bearing ratio and Texas triaxial, which have been used to evaluate marginal base course materials in Western Australia for use as surface courses under light seal coats. It is also used to derive an elastic modulus and to determine variability. Strength changes during and after construction, in terms of the impact value obtained from this test, are examined. From these observations, a procedure is suggested for selection of base course materials in the field. It is concluded that the effect of traffic compaction on strength improvement is worthy of further investigation and evaluation.

The selection of materials for base courses for low-volume roads involves to a large degree strength evaluation in relation to the in-service moisture environment. For "wet" climates this has led to testing "saturated" or "soaked"; for "dry" climates, testing after "drying back" from compaction moisture content has become common. In the latter situation, Australian practice is now related to the establishment of a design moisture content (DMC) that is based on an equilibrium moisture content (EMC) (1). By coupling DMC with the expected density, a design strength is established from laboratory tests, usually in terms of California bearing ratio (CBR). The procedure of using in situ strength testing at expected moisture-density conditions is provided for, particularly when there are similar existing pavements that can be used as a basis for comparison. The most common test used in this respect is the cone penetrometer (2). Field CBR testing has