Waterbound Macadam as a Base and as a Drainage Layer

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ABSTRACT

The good performance of this large single-sized granular-base type in wet regions has revived interest in this once labor-intensive form of base construction. Specifications have been improved and guidelines set for the mechanized construction of waterbound macadam bases. Density standards have been set for typical waterbound macadam bases following the construction of four experimental sections, the development of the Rondawel density test, and accelerated testing by the heavy vehicle simulator (HVS). The performance of these waterbound macadam bases is compared with that of other high-quality granular bases. Guidelines are set for effective elastic moduli for typical waterbound macadam bases and the mechanistic analysis of such granular-based pavements. Waterbound macadam-based pavements are also evaluated as a drainage layer and recommendations are made for improvement.

Waterbound macadam is an old construction technique originating with John Loudon McAdam. The single-sized coarse aggregate is placed and compacted separately on a prepared subbase before the voids are filled with fines, and the material is then compacted and slushed, hence the name waterbound macadam (1).

A considerable amount of waterbound macadam base construction has been done in South Africa, chiefly in the major metropolitan areas. This type of construction reached its climax in the period 1945 to 1955, after which it declined in popularity, primarily because of the high cost of labor-intensive construction. Over the years, however, waterbound macadam bases have distinguished themselves as granular bases with excellent performance. It is particularly in the wet regions of South Africa that waterbound macadam bases have proved to withstand the destructive influence of water and heavy traffic better than other granular base pavements (1). Roads with waterbound macadam bases have shown virtually no signs of shear deformation in wet conditions even after 30 to 40 years of use. For this reason there has been revived interest in waterbound macadam base construction, and the local road authorities and the National Institute for Transport and Road Research (NITRR) cooperated to construct four experimental sections of waterbound macadam near Marianhill, on Main Road 85 (2) and on National Route 3/1 (3). The aim of the work was to establish guidelines for the construction of waterbound macadam bases and to test these sections with the accelerated testing facility, the heavy vehicle simulator (HVS) (4), developed by the NITRR.

Advances made in the specification of materials for waterbound macadam and the objective control of the construction technique are discussed briefly. The accelerated testing of waterbound macadam bases made it possible to evaluate them as structural layers and facilitated the determination of ranges of elastic moduli for the mechanistic analysis of such pavements. Waterbound macadam was also evaluated as a drainage layer during the accelerated testing (1) by monitoring the effects of water introduced into the layer.

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MATERIAL CHARACTERISTICS

Waterbound macadam can be defined as a high-quality granular base. The material used should be freshly crushed hard rock. The normal NITRR specification (5) is set out in terms of grading, Atterberg limits, crushing strength, flakiness index, water absorption, and conductivity. It is, however, the grading of waterbound macadam that distinguishes it from other high-quality granular bases (1). The single-sized grading of the coarse aggregate and the maximum stone size of 53 to 75 mm are the factors that ensure granular interlock with high resistance to shear failure. Typical recommended gradings are shown in Table 1.

TABLE 1 Typical Coarse Aggregate Gradings for Waterbound Macadam

Sieve Size (mm)	Percentage Passing by Mass		
	No. 1	No. 2	No. 3
75	100	100	100
53	85-100	85-100	85-100
37.5	35-70	0-30	0-50
26.5	0-15	0-5	0-10
19	0-10		

Although it could easily be defined as an open-graded base (6), waterbound macadam requires that the voids be filled (with fines). Fines contribute mainly to stability and provide some cohesion in the base material. The plasticity index of the fines is limited to 6 percent. The grading merely serves to ensure the free flow of the fines in the dry state to fill the voids between the coarse aggregate. The grading is as follows:

Sieve Size (mm)	Percentage Passing by Mass
9.5	100
4.75	85-100
0.075	10-25

This allows freedom in regard to the grading, but the influence of grading on the permeability of such

a base layer should not be underestimated, as discussed later.

ADVANCES IN CONSTRUCTION CONTROL AND SPECIFICATIONS

Four experimental sections with waterbound macadam bases were constructed in the vicinity of Pinetown, Natal. The pavement structures all had broken rock subgrades in cuttings where drainage problems normally occur. Adequate subsurface drainage was installed (2). All four experimental sections had a 300-mm thick cemented subbase of a high quality (C3, see Table 2) (7). The waterbound bases were either 150, 170, or 250 mm thick with no-fines concrete edge restraints encasing drainage pipes. The asphaltic-concrete wearing course was 50 to 70 mm thick and the experimental sections were either 100 or 150 m long (3).

TABLE 2 Definition of Material Symbols Used in Catalogue Designs

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense-graded unweathered crushed stone; Max. size 37,5 mm. 86-88 % of apparent density. Fines PI $\leq$ 4
	G2	Graded crushed stone	Dense-graded unweathered crushed stone; Max. size 37,5 mm 100-102 % mod. AASHTO. Fines PI $\leq$ 6
	G3	Graded crushed stone	Dense-graded stone + soil binder; Max. size 37,5 mm; Minimum 98% mod. AASHTO. Fines PI $\leq$ 6
	G4	Natural gravel	CBR $\leq$ 80; PI $\geq$ 6
	G5	Natural gravel	CBR $\leq$ 45; PI $\geq$ 10 to 15 depending on grading; Max. size 63 mm.
	G6	Natural gravel	CBR $\leq$ 25; Max. size $\geq \frac{2}{3}$ layer thickness
	G7	Gravel-soil	CBR $\leq$ 15; Max. size $\geq \frac{2}{3}$ layer thickness
	G8	Gravel-soil	CBR $\leq$ 10; at in situ density
	G9	Gravel-soil	CBR $\leq$ 7; at in situ density
	G10	Gravel-soil	CBR $\leq$ 3; at in situ density
	BC	Bitumen hot-mix	Continuously-graded; Max. size 26,5 mm.
	BS	Bitumen hot-mix	Semi-gap-graded; Max. size 37,5 mm.
	TC	Tar hot-mix	As for BC (continuously-graded)
	TS	Tar hot-mix	As for BS (semi-gap-graded)
	PCC	Portland cement concrete	Mod. rupture $\leq$ 3,8 MPa; Max. size $\geq$ 75 mm
	C1	Cemented crushed stone or gravel	UCS 6 to 12 MPa at 100% mod. AASHTO; Spec. at least G2 before treatment; Dense-graded.
	C2	Cemented crushed stone or gravel	UCS 3 to 6 MPa at 100% mod. AASHTO; Spec. generally G2 or G4 before treatment; Dense-graded.
	C3	Cemented natural gravel	UCS 1,5 to 3,0 MPa at 100% mod. AASHTO; Max. size 63 mm.
	C4	Cemented natural gravel	UCS 0,75 to 1,5 MPa at 100% mod. AASHTO; Max. size 63 mm.
	AG	Asphalt surfacing	Ref. TRH8 gap-graded
	AC	Asphalt surfacing	Ref. TRH8 continuously-graded
	AS	Asphalt surfacing	Ref. TRH8 semi-gap-graded
	AO	Asphalt surfacing	Ref. TRH8 open-graded
	S1	Surface treatment	Ref. TRH3 single seal
	S2	Surface treatment	Ref. TRH3 multiple seal
	S3	Sand seal	Ref. TRH3
	S4	Cape seal	Ref. TRH3
	S5	Slurry	Fine grading
	S6	Slurry	Coarse grading
	S7	Surface renewal	Rejuvenator
	S8	Surface renewal	Diluted emulsion
	WM1	Waterbound macadam	Max. size 75 mm, PI of fines $\geq$6 88-90% of apparent density
	WM2	Waterbound macadam	Max. size 75 mm, PI of fines $\geq$6 86-88% of apparent density
	PM	Penetration macadam	Coarse stone + keystone + bitumen or tar
	DR	Dumprock	Ungraded waste rock, max. size $\frac{2}{3}$ layer thickness
	CB	Concrete paving blocks	Wet crushing strength $\leq$ 30 MPa; interlocking shapes

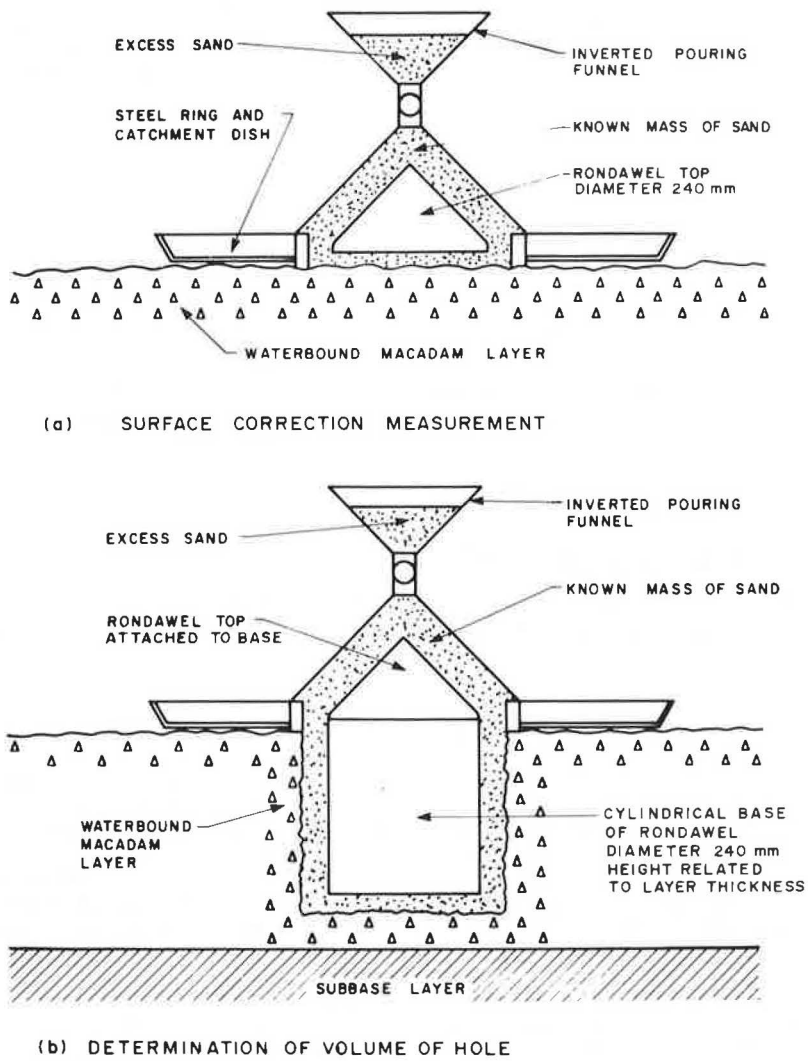


FIGURE 1 Rondawel test procedure.

The construction technique developed by a process of evolution from labor-intensive to mechanized methods (2,3). The spreading or placing of the coarse aggregate was done, for example, by a heavy grader and the fines were spread with a stone spreader. Alternative means of placing the coarse aggregate include converted bitumen and concrete pavers, spreading boxes towed behind tractors or trucks, and even drip spreaders. These changes made the alternative of waterbound macadam-based construction economically viable (8).

Quality control during construction was advanced by the development of a large sand replacement density test, the Rondawel test (1). This test is shown in Figure 1. The test enabled specifications for waterbound macadam to move away from a recipe method to some measure of objective regulation of construction control. Resulting from the accelerated testing of the experimental sections with the HVS, apparent density specifications could be established, as follows (E80 = equivalent 80-kN axle repetitions):

Traffic (E80's)	Waterbound Macadam Category	Apparent Density (%)
Up to 3 x 10^6	WM2	86-88
3-50 x 10^6	WM1	88-90

The traffic classification is an oversimplification of the traffic classes and categories given in the NITRR document on pavement structural design, TRH4 (7), indicating that higher density bases are required for higher traffic classes and road categories. This summary specification is shown in Table 3 with the material definition or classification system used in the catalogue of designs in TRH4 (7).

WATERBOUND MACADAM AS A GRANULAR BASE

In order to evaluate waterbound macadam as a base course, six HVS tests to date have been completed on the four experimental sections. In each HVS test a large number of specialized measurements of the

TABLE 3 Porosity Values and Indications of Permeability for Waterbound Macadam Bases

Material Description	Porosity (n) (%)	Indications of Permeability (L/hr)
Typical coarse WM[a] aggregate alone	25-40	> 1.1
WM bases (average)	13.5	1.5
WM base (fines of windblown sand)	21.6	100
WM2 base	11	1.27
WM1 base	3	0.25

[a]As for unconsolidated gravel deposits *(13,6)*.

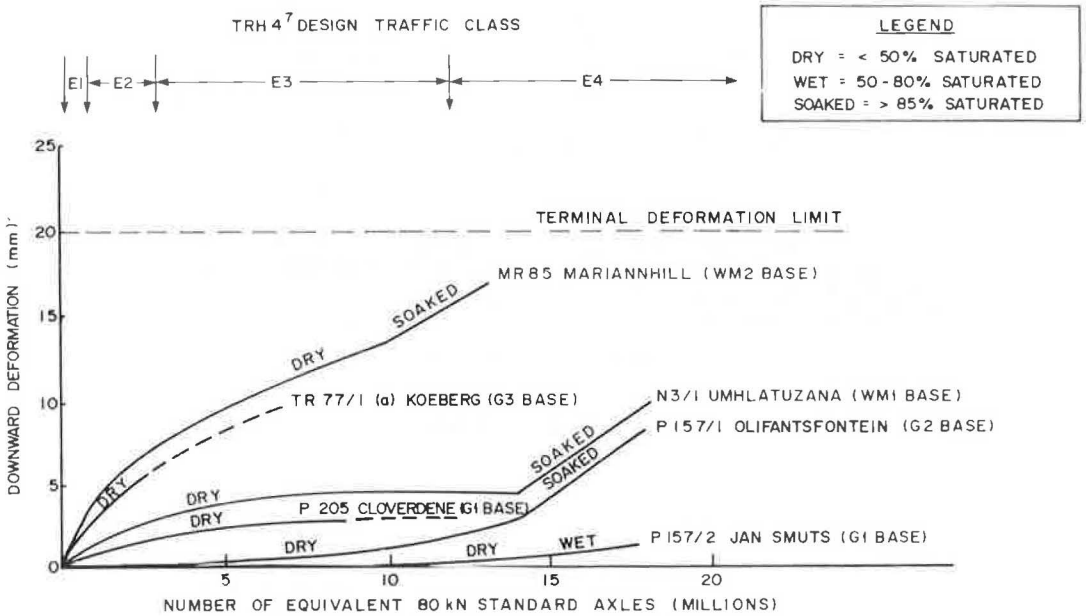


FIGURE 2 Comparison of the pavement deformation induced in various crushed-stone base pavements by HVS trafficking.

pavement response and behavior were taken. These measurements are described elsewhere in detail (4).

One of the measurements taken during such an HVS test is the permanent deformation. This is measured on the surface by means of a fully automatic, motor-driven profilometer that tracks over the road surface or manually by means of a 2-m straight edge. The deformation originates in the granular base. This is true for granular-based pavements with well-protected subgrades (9) and can be confirmed by profile trenching across a test section after completion of such a test.

The terminal rut (permanent downward deformation) limit for higher-category roads is 20 mm. The number of repetitions of the dual wheel load used in any test is calculated and expressed in terms of the number of standard 80-kN axle repetitions (E80's). In order to compare the deformation of different granular-based pavements tested, a damage coefficient (d) equal to 3 is used in the relative damage formula:

$$D = (P/80)^d \quad (1)$$

where

D = relative damage value,
P = axle load (kN), and
d = damage coefficient.

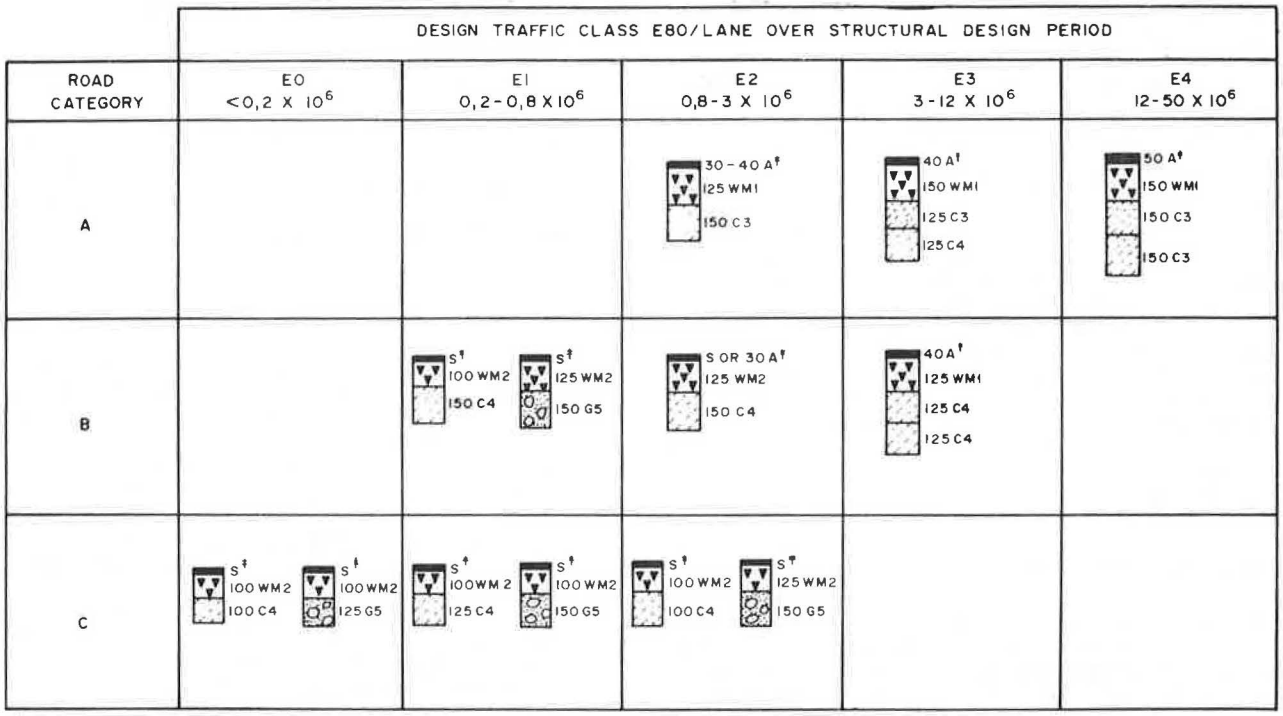
A value of d equal to 3 rather than 4.2 was found to be applicable to deep pavement structures with granular bases (9). In Figure 2 a comparison is made of various granular-based pavements tested with the HVS. The behavior of typical WM1 and WM2 quality bases is compared with the findings of Maree et al. (9).

The WM1 and WM2 bases show the typical rapid initial deformation of a granular-based pavement. In WM1, which has the higher density, this initial deformation is less than in the lower-density WM2 base. This indicates that even higher densities could be specified, which would reduce this tendency to rapid initial deformation. After the settling-in period, the rate of increase in deformation becomes constant for the WM1 base and to a lesser extent for the WM2 base, as for other high-quality bases, such as the G1 and the G2. The influence of water will be discussed in the next section; the discussion in this section is limited to the behavior of dry, well-drained bases. The highest traffic classes according to the TRH4 classification (7), even 50 x 10^6 E80's, are allowed for in the design with a WM1 base, with the proviso that environmental influences must be neutralized by an adequate maintenance program (9). The WM1 base compares well with other high-quality granular bases such as the G2 and even the G1. The WM2 base compares well with the weaker-quality granular base, the G3.

The second measurement used to indicate the structural capacity of a waterbound macadam base pavement is the elastic deflection measured at different depths within the pavement. The instrument used is the multidepth deflectometer (MDD) (10). Effective elastic moduli were determined by means of a curve-fitting iteration procedure using the MDD deflections (1). Waterbound macadam is a typical stress-dependent granular material; the effective elastic moduli increase with the applied wheel load, and density under trafficking increases as well (1,11). The recommended range of effective elastic moduli for waterbound macadam base layers, as measured and calculated, is as follows for various types of subbase support:

Material Class	With Stabilized Subbases (MPa)	With Granular or Cracked Stabilized Subbases (MPa)
WM1	150-700	100-400
WM2	120-400	70-250

The stress states in the granular waterbound macadam layers were calculated on the basis of these effective elastic moduli by using a linear elastic layer program developed by the NITRR in 1977 (12). No laboratory triaxial shear-strength parameters [C (apparent cohesion) and ϕ (angle of internal friction)] of waterbound macadam have yet been determined, and therefore the values for a G1 base in the wet to saturated states were assumed as a conservative guide (1) in the calculation of the safety factor (11). The maximum stone size for the G1 material is 37.5 mm.



†SYMBOL A DENOTES AG, AC OR AS. SYMBOL S DENOTES S2 OR S4

FIGURE 3 Waterbound macadam bases (for explanation of symbols, see Table 2).

A safety factor (f) used in the mechanistic analysis of granular bases to predict their structural behavior is calculated as follows:

$$F = k\{\sigma 3|\tan^2 [45 + (\phi/2)] - 1| + 2C\tan[45 + (\phi/2)]\}/(\sigma_1 - \sigma_3) \quad (2)$$

where

C = apparent cohesion,
ϕ = angle of internal friction,
σ_1, σ_3 = major and minor principal stresses, and
k = constant, depending on the moisture conditions.

An analysis of a typical waterbound macadam base pavement in the wet to saturated states led to the recommendation and incorporation of waterbound macadam bases for wet regions in the new (1984) catalogue of designs (7). These designs are shown in Figure 3, and the material classification is summarized in Table 2. It is recommended that WM1 bases with stabilized subbases be used for the higher road categories and the higher traffic classes, and granular subbases with WM2 bases can be used for the lower road categories and traffic classes (1).

WATERBOUND MACADAM AS A DRAINAGE LAYER

The emphasis in the construction of waterbound macadam was on achieving a dense granular base with a high shear force resistance due to the coarse granular interlock. The history of waterbound macadams in South Africa (1) and abroad (6) has shown, however, that an open-graded granular base of this kind also provides an excellent drainage layer. The addition of fines to fill the voids between the coarse aggregate reduces the drainage capabilities of such a layer.

Although the experimental sections of waterbound macadam were not designed according to the drainage principles suggested by Cedergren (6), the influence of water on these waterbound bases was monitored in various ways. During all HVS testing on the experimental sections, water was introduced into the WM1 and WM2 bases by means of a system of perforated pipes. The 38-mm diameter pipes were installed right on the higher edge of the trafficked test section (8 m long and 1 m wide), thereby ensuring that water flowed down the cross and longitudinal gradients through the base of the trafficked section. Water was administered normally at atmospheric pressure, and the rate was measured. In one test the water was administered at a pressure head of 1 to 2 m. Water was normally administered toward the end of a test, although in some cases this was done throughout the test.

The typical deformation behavior of a granular-based pavement when the base is soaked is shown in Figure 2. There is a sharp increase in deformation with the increase in trafficking, because of the development of a state of excessive pore-water pressure. This leads to erosion of fines, deformation, and potholing. When a typical WM2 base has adequate drainage, however, there is virtually no change in the rate of deformation. When water is introduced into a WM1 base at a pressure head of 1 to 2 m, it is possible to develop a state of excessive pore-water pressure under trafficking, showing the typical increase in deformation. Under such aggressive soaked conditions the fines are again washed through cracks in the surfacing layer or into untrafficked parts of the test section, where they accumulate between the surfacing and base layer. Shear deformation has not occurred because the granular interlock of the coarse aggregate matrix is still intact.

Indications of permeability of the exposed surface of the waterbound macadam base were measured by means of the in situ falling-head permeability test (13). Standard material tests and density determinations of the waterbound macadam bases made it possible to calculate the porosity (n), which can have an important controlling influence on permeability (14).

Table 3 gives a summary of these results as measured on various waterbound macadam bases.

It is clear from Table 3 that although the addition of fines to the voids of the coarse aggregate leads to a reduction in porosity, the permeability values still indicate an efficient drainage layer. The fact that the small particles influence permeability more than the large particles (15) is clearly illustrated, too. When the fines used to fill the voids are of windblown single-sized sand, porosity values nearly double and the indications of permeability increase 100-fold. This indicates that if the percentage of fines passing the 0.075-mm sieve could be limited to less than 10 percent, the permeability of waterbound macadam bases could be increased significantly. The increase in density from a WM2 standard to a WM1 standard leads to a reduction in porosity and indication of permeability values. In such a dense state the base is comparable with other dense, high-quality granular bases, which are virtually impermeable.

SUMMARY AND CONCLUSIONS

1. The single-sized grading and maximum stone size (75 to 53 mm) of the coarse aggregate characteristic of waterbound macadam material primarily ensure granular interlock and high resistance to shear failure.
2. The possibility of mechanizing the construction of waterbound macadam bases was evaluated by departing from labor-intensive techniques during the construction of the four experimental sections.
3. Quality control and objective specification criteria have been enhanced by the development of the Rondawel density test and the establishment of recommended density specifications in terms of apparent density.
4. The performance of waterbound macadam bases is typical of granular bases when permanent deformation behavior during accelerated testing is compared. Initial density has a direct influence on the rapid initial deformation.
5. The determination of effective elastic moduli by MDD deflection back-calculation procedures makes it possible to establish ranges of effective elastic moduli. Suggestions for the mechanistic analysis of such bases in relation to the model used for granular materials have been made. Typical catalogue designs for waterbound macadam-based pavements for wet regions have been recommended and incorporated in the NITRR's pavement design document, TRH4 (7).
6. Waterbound macadam, properly constructed to WM1 density and with adequate support, can carry the highest traffic classes, bearing even 50 x 10^6 E80's. There is even scope for an increase in the density specification that would offset initial deformation tendencies under trafficking. The stress-dependent material has high resistance to shear failure, which means a better quality of granular base as density increases under trafficking.
7. Waterbound macadam has the potential to be a very efficient drainage layer, especially if coarser fines are specified. Higher densities, however, mean lower permeability. On this point the structural requirements and those for a drainage layer may oppose each other, but the existing density specifications (WM1 and WM2) appear to satisfy both requirements adequately.
8. The importance of providing drainage for any base layer is emphasized by the behavior of waterbound macadam bases. Even in such high-quality granular bases, excessive water in the base can lead to excessive pore-water states and a pumping of fines from any erosion-susceptible layer in the pavement structure.

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Cost-Estimating Model for Low-Volume Roads

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ABSTRACT

The Forest Service, U.S. Department of Agriculture, is required to perform accurate and comprehensive road cost estimates to carry out the legislative intent of Congress in the programming, allocation, and use of funds. This study utilizes multiple regression analysis to develop unit-price equations and total project cost equations for cost estimation. A sample consisting of 26 projects from the western United States is used for preliminary model development. The equations developed are applied to a second sample with six projects located in the same area. The results indicate that the model has potential for determining reliable preliminary road cost estimates. Because of its simplicity, this model could reduce the resources spent on this task and lead to the reduction of transportation cost.

One of the major concerns of the Forest Service, U.S. Department of Agriculture, is the accuracy of road cost estimates. Estimates are used to carry out the legislative intent of Congress in the programming, allocation, and use of funds. Two types of preliminary estimates used for this purpose are the office estimate and the field-verified estimate. The former is based on office information such as land use plans, aerial photographs, topographic maps, and other resource information. It is used to support activities such as land use planning, resource management planning, area transportation planning, and long-range (over 5 years) fiscal programming. The second type of estimate is based on all the information available for an office estimate plus more extensive field verification, including some rough field measurements and more detailed resource information gathering. This estimate is used in resource and transportation project planning, short-range (2 to 5 years) fiscal programming, and budgeting.

The accuracy of both preliminary estimates varies in accordance with the reliability of the data base. Deviations can range from 35 to 50 percent for the office estimate and from 20 to 30 percent for the field-verified estimate (1). Two main sources of these deviations are unit-quantity and unit-price predictions. The major concern of this study is unit-price prediction.

Conventionally, road costs are estimated by either constructed costs or historical bids or a combination of both. The constructed-cost method utilizes production rates, labor and equipment costs, profit and risk, taxes, and material costs to estimate the unit price. On the other hand, the unit price derived from the historical-bid approach is estimated by the weighted average of bids submitted by contractors over some period of time. These unit prices are adjusted by a cost trend factor to reflect the cost at the time when the project will most likely be constructed.

The objective of this study is to use regression analysis to develop unit-price estimating models based on historical-bid data. Several other studies have been made along these lines to improve cost estimation (2,3). The results of these studies indicated that by using regression analysis, it is possible to estimate highway construction costs with a higher degree of reliability than can be obtained by simple unit-cost weighted averages. In the present study, a sample of 26 new construction projects was utilized for model development. This model was verified by six projects, including new construction and reconstruction. However, it should be noted that this paper does not suggest weakness or deficiency in current Forest Service policies or practices but is intended to illustrate the potential usefulness

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